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WATER SUPPLY
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WATER SUPPLY ENGINEERING

By

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and

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FOURTH EDITION

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WATER SUPPLY ENGINEERING

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IV

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PREFACE TO THE FOURTH EDITION

In preparing the text for the fourth edition the conflicting demands to make the text brief, to retain everything of value already in the text, and to add recent valuable contributions to waterworks knowledge presented a problem difficult to solve with satisfaction. The authors' dilemma lay in choosing between an encyclopedia and a primer, neither of which would be satisfactory. A solution has been attempted in the emphasis of functional rather than structural procedure and practical applications rather than theoretical design. The discussion of theories available in more theoretical textbooks has, in general, been removed. These omissions include much of the theoretical development of formulas in the mathematics of finances, of the hydraulics of the flow of water, of steam and reciprocating pumping machinery, of electricity, and of the epidemiology of water-borne diseases. In their place appear applications of such theories to practice in finances, hydraulics, pumping machinery, electrical equipment, and water purification. The aim, in general, has been to apply theories and fundamentals to practice in the waterworks field. Revisions and additions have been made to emphasize the timeliness and utility of the information presented.

HAROLD E. BABBITT
JAMES J. DOLAND

URBANA, ILL.
March, 1949

PREFACE TO THE FIRST EDITION

This book is intended primarily as a textbook for use in civil engineering courses as presented in many engineering college curricula. The authors have had the opportunity of presenting portions of the manuscript to their classes at the University of Illinois. In this presentation the need for a comprehensive and up-to-date text has been felt for years. The literature of waterworks is rich, and available textbooks covering such fundamentals as history, finances, hydraulics, etc., are unsurpassed. The use of such books for the teaching of fundamentals, supplemented by references to current literature, is laborious to the instructor, time consuming for the student, and unsatisfactory to all concerned. A textbook on waterworks, to be of continued value, must receive frequent revisions to keep pace with the rapid changes in practice. In preparing this book the authors have tried to combine both fundamentals and practical procedure in a single volume within the limitations of space and cost customary in engineering college texts in specialized professional courses.

Limitations of space have prevented the exhaustive treatment of all the subjects mentioned in the book. An exhaustive treatment of the subject of almost any chapter of the book would require a volume as large as or larger than this entire book. The design of dams is one subject worthy of greater space than is allotted to it. But few, if any, earlier textbooks on waterworks devote space to the subject of electrical equipment. Knowledge of the application of electricity to waterworks practice is not easily acquired by civil engineers. It is felt, therefore, that this subject should be presented for their benefit. The chapter on The Utilization of Electricity has been presented for students who have completed a study of the subject of electricity and magnetism in college physics and who have some acquaintance with the subject of electrical engineering. The space devoted to the subject is inadequate, but it has been filled with the idea that half a loaf is better than none. The textbook is not intended as a comprehensive treatise on waterworks or as a handbook. Such volumes are of little value as textbooks. In this book the student is introduced to the subject, fundamentals are stated, practice is explained, and, where desirable, references are made to articles of value on the subject. The problems presented in Appendix II should be found helpful to the teacher.

The value of a textbook in professional subjects is measured partly by its value to the engineer practicing in the same field. No book which treats the subject in an exclusively academic manner can be successful as a textbook. In this book an attempt has been made to illustrate fundamentals with practical applications thereof. Where recognized practice is based on empiricism, the procedure is explained and practical illustrations are presented.

An attempt has been made to give credit to all sources from which material has been taken. The authors apologize for any omissions of this character which may, unknowingly, have occurred. Thanks are due to the American Water Works Association, to the *Engineering News-Record,* and to others for their permission to quote from their publications.

The authors are greatly indebted to W. D. P. Warren and to Alex Van Praag, Jr., of the firm of Warren & Van Praag, consulting engineers of Decatur, Illinois, for their comprehensive and exhaustive review of the manuscript and for their valuable suggestions and contributions. Thanks are due to Prof. A. R. Knight, assistant professor of electrical engineering at the University of Illinois, for his review of the chapter on The Utilization of Electricity, and to W. N. Espy, associate in mechanical engineering at the University of Illinois, for his review of the chapters pertaining to mechanical engineering. The permission given by Dr. Elwood Mead, Commissioner of the United States Bureau of Reclamation, to reproduce the information taken from the standards of engineering practice in the Bureau is gratefully acknowledged.

HAROLD E. BABBITT
JAMES J. DOLAND

URBANA, ILL.
December, 1929

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CHAPTER I

INTRODUCTION

1. Historical.[1] A knowledge of the history of the development of waterworks is desirable to emphasize changes in practice and the relatively recent development of present-day waterworks methods and equipment. The history of waterworks is as ancient as the history of man. Waterworks structures are found in the excavation of prehistoric ruins. The remains of Lake Moeris, in Egypt, indicate its construction about 2000 B.C. It was the largest of the reservoirs of the Nile Valley, which supported 20,000,000 people—four times the present-day population.

> In ancient times the valleys of the Euphrates and Tigris, now almost a desert, were densely populated. Four thousand years ago the rulers of Assyria had converted these sterile plains and valleys into gardens of extreme productiveness by the construction of immense artificial lakes for the conservation of the flood waters of the river, and great distributing canals for irrigation. One of these canals, the Nahrawan Canal, supplied by the Tigris, was over four hundred miles long and from two hundred to four hundred feet broad, with sufficient depth for the navigation of vessels of that time.
>
> In India, tanks, reservoirs and irrigating canals were constructed many centuries before the Christian Era and a great part of that country was kept in the highest state of cultivation. Some of the tanks or artificial lakes covered many square miles and were often fifty feet or more in depth.
>
> Evidences exist in New Mexico and Arizona that in prehistoric times a race, now extinct, had extensive irrigation works and cultivated large areas.

Biblical references to waterworks are frequent. For example, in II Kings 20:20:

> And the rest of the acts of Hezekiah, and all his might, and how he made a pool, and a conduit, and brought water into the city, are they not written in the book of the chronicles of the kings of Judah?

And the reference to Elisha's purification of water with a coagulant which is stated in II Kings 2: 19–22.

The water supply for the city of Rome is one of the marvels of ancient times. The water was brought from surrounding hills in aqueducts totaling about 381 miles in length. The first aqueduct, the Appia, was

---

[1] See also BAKER, M. N., "The Quest for Water," American Water Works Association, 1948.

10 miles long and was built in 312 B.C. The Claudia Aqueduct is illustrated in Fig. 1. All these aqueducts were constructed along the hydraulic grade line in order to avoid the necessity for building pressure conduits. Iron pipe was unknown at the time. Lead was the only material available to carry water under pressure. As lead was not suitable for high pressures, it was necessary to convey water in aqueducts at atmospheric pressure.

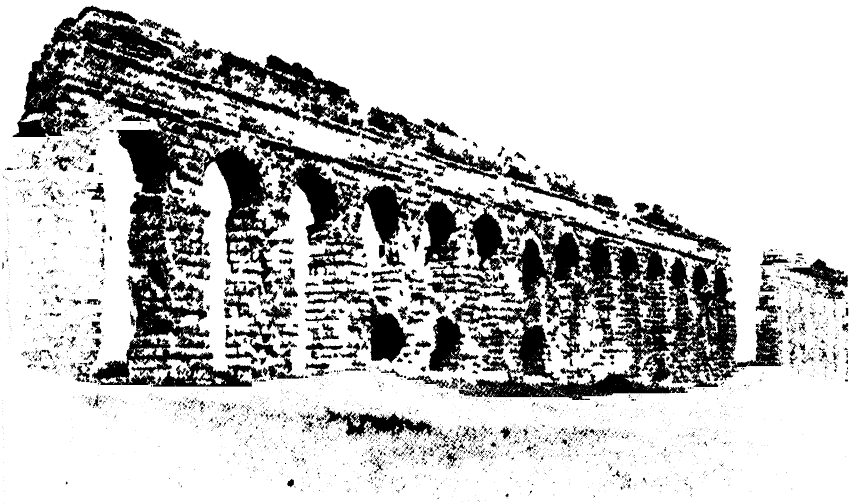

FIG. 1.—A Roman aqueduct. Claudia of dimension stone, and Novus of brick and concrete (*From "Frontinus and the Water Supply of Rome," by Clemens Herschel.*)

Much detail concerning the history of the water supply of Rome is presented in "Frontinus and the Water Supply of Rome," translated by Clemens Herschel. Frontinus was water commissioner for the city of Rome about A.D. 100 under the Emperor Nerva. His records are a most human document, revealing the intimacies of the life of the people of Rome and relating his difficulties with human nature which was prone to err then as it is today. He had considerable difficulty with persons who stole water from the aqueducts:

> The cause for this is the fraud of the water-men whom we have detected diverting water from the public conduits for private use; but a large number of proprietors of land also, whose fields border on the aqueducts, tap the conduits; whence it comes that the public water courses are brought to a standstill by private citizens, yea, for the water of their gardens.

The baths constructed by the Romans are among the outstanding features of their era. The baths of Bath, England, illustrated in Fig. 2, are in an excellent state of preservation. Natural hot water still flows through the original pool. It is supposed that these baths were constructed during the first century A.D., possibly when Frontinus was governor of Britain, probably before his position as water commissioner of Rome.

Evidences of ancient waterworks that were constructed in the Western Hemisphere are to be found in Arizona, New Mexico, Central America, and the northern parts of South America. The Cenote, or Great Pool of Sacrifice, was constructed at Chichen Itzá, Yucatán, Mexico. No legible record has been left by the people who constructed this pool. It is possible that much of the work of the prehistoric races in Central America was done during the Christian Era, but proof to that effect is not certain. The aqueduct of Zempola, in Mexico, was constructed about A.D. 1560. It was about 28 miles long and supplied the city of Otumba.

FIG. 2.—Roman baths at Bath, England.

During the Middle Ages in Europe there was more destruction than construction. It is probable that the use of polluted water was the cause of many of the epidemics that swept the continent. All was not destruction, however, as some aqueducts were constructed in Spain by the Moors during the ninth century, and the Roman aqueduct at Seville was repaired in 1172. London and Paris increased the capacity of the public water supplies. A small aqueduct was constructed in Paris in 1183, and water was brought to London from the outlying districts by means of lead pipes and masonry conduits in 1235. Cast-iron pipe was laid in France during the reign of Louis XIV to supply water to the fountain at Versailles.

The modern renaissance in waterworks construction is marked at the start of the nineteenth century by the appearance of steam-driven pumping machinery and of cast iron. Previous to the appearance of these features, a general public water supply with a service connection to each house was unknown. Wood and lead were the only materials available for pipes, and, as then constructed, the pipes would withstand but little internal pressure. The first steam-driven pumping engine is said to have been installed in London in 1787.[1] Previous to this, pumps driven by the river current had been used. The first steam pumping engine in the United States was installed in Philadelphia in 1800. Cast-iron pipe was laid in Philadelphia in 1804 and in London, England, in 1807. "Public water supplies in the United States date from 1652 at Boston, about 1732 at Schaefferstown, Pa., and 1761 at Bethlehem, Pa."[2]

2. Development of Water Purification. The first steps in water purification were probably taken in China and India thousands of years ago. For centuries it has been the practice of the Chinese to put alum in tubs of water to clarify it. The same thing was done in Egypt:[3]

> In the "Ousruta Sanghita"—a collection of medical lore in Sanskrit, probable date 2000 B.C., Chap. XIV, verse 15—appears this instruction: "It is good to keep water in copper vessels, to expose it to sunlight, and filter through charcoal." In the "Neghrund Bhusan"—a collection of medical maxims from the "Ayura Veda," the earliest Sanskrit work on medicine extant, of about the same date in the chapter on water, in the last sloka but two, it is directed to treat foul water by boiling and exposing to sunlight and by dipping seven times into it a piece of hot copper, then to filter and cool in an earthen vessel.

An early reference to water purification occurs in the Bible, as follows:[4]

> 19. And the men of the city said unto Elisha, Behold, I pray thee, the situation of this city is pleasant, as my lord seeth; but the water is naught, and the ground barren.
>
> 20. And he said, Bring me a new cruse, and put salt therein. And they brought it to him.
>
> 21. And he went forth unto the spring of the waters, and cast the salt in there, and said, Thus saith the Lord, I have healed these waters; there shall not be from thence any more death or barren land.
>
> 22. So the waters were healed unto this day, according to the saying of Elisha which he spake.

There are but few records of advance in water purification up to the introduction of the slow sand filter or the "English system," which dates back to A.D. 1829. The first filter was constructed during this year by

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[1] KIRBY, R. S., and P. G. LAURSON, "Modern Civil Engineering," p. 191, Yale University Press, New Haven, 1932.

[2] "Water Works Practice," p. 3, American Water Works Association, 1925.

[3] *Water*, Aug. 15, 1905.

[4] II Kings 2: 19–22.

James Simpson. In 1839 the London waterworks introduced the first filters, and in 1853 "English" filters were constructed in Berlin.

The treatment of water to remove pathogenic organisms had its beginnings about the middle of the nineteenth century, after Koch had made known his spectacular findings. Filtration of water for the London metropolitan district was made compulsory in 1855, and at about the same time the so-called English filters were installed elsewhere in England and on the Continent. The filters at Altona, Germany, were installed about 1852.

The findings of James B. Kirkwood, who visited England and Germany in 1866–1868 for the city of St. Louis, resulted in a marked advance in American practice in water purification. One of the earliest filters built in the United States was that at Poughkeepsie, N.Y., in 1870 In 1883 Prof. William Ripley Nichols published the first authoritative book on the subject of water purification, as a result of his European visit in 1878. Early American experiments in water purification and sewage treatment were started in 1887 at the Lawrence Experiment Station of the Massachusetts State Board of Health. All these experiments, constructions, and studies were along the line of slow sand filtration.

The first filter plant for a public water supply in which alum was used as a coagulant was constructed at Somerville, N.J., in 1884; in the same year Alpheus Hyatt secured a patent for the process. A notable step in the advance of scientific knowledge of rapid sand filtration was completed with the work, in 1895 and 1897, of Charles Hermany and G. W. Fuller, at Louisville, Ky. The principles established there have served as the basis for the design of mechanical filters that have since been constructed in the United States.

The first important mechanical filtration plant to be constructed along modern lines was at Little Falls, N.J., in 1902.[1] The increase in the number of filtered public water supplies has been rapid since 1901, when the Hyatt patent expired, until today the existence of water purification in a city is more common than its absence. In 1925 there were about 587 rapid sand filter plants and 47 slow sand filter plants in the United States. The rapid sand plants had a total capacity of 4,048 m.g.d. gal. per day and served 18,610,000 persons. The slow sand filter plants had a total capacity of 916 million gallons per day (m.g.d.) and served 5,054,000 persons.[2] In subsequent years the number of rapid sand filter plants has increased greatly, and the number of slow sand filter plants has decreased. In 1942, Weibel[1] reported 74,308,000 persons in urban districts served by public water supplies. There were 98 slow sand filter plants with a total capacity of 482 m.g.d., serving 3,579,200 persons; and there were 1,535 rapid sand filter plants with a total capacity of 2,600 m.g.d., serving 22,921,600 persons.

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[1] Whipple, G. C., "History of Water Purification," *Proc. A.S.C.E.*, Vol. 47, p. 654, 1921.

[2] Eddy, H. P., "Historical Review of Sanitary Engineering in the United States," *Proc. A.S.C.E.*, 1927, p. 1587.

3. Importance of Waterworks. Water is the most essential commodity, other than air, for the continuation of life. Without it man quickly dies. The existence of the great concentrations of population in our present-day cities and the pursuit of the myriads of activities therein would be impossible without the availability of water. A city with a restricted water supply is a city of restricted growth.

As the population of a community increases, the need for a public water supply is felt. Dependence can no longer be placed on private wells. The concentration of population makes satisfactory well sites more difficult to locate, and the quality of the water becomes more uncertain. Among the greatest benefits to be secured from a public water supply is fire protection. Protection against disastrous conflagrations is a potent factor in convincing a community of the need for the installation of the public water supply. A large percentage of the cost of waterworks is chargeable to fire protection, as is shown in Table 1.

TABLE 1.—PROPORTION OF COST OF WATERWORKS CHARGEABLE TO FIRE PROTECTION
(Estimated by the National Board of Fire Underwriters.)

Population, thousands	Percentage of the total cost of the waterworks which is chargeable to fire protection	Percentage of the cost of the distribution system which is chargeable to fire protection
10	60	56
50	32	29
100	23	17.5
300	13	4

4. Reliability of Waterworks. Because of the importance of waterworks and the great dependence of the public on an unfailing supply, it is necessary to design a waterworks system of unquestioned reliability, even though considerable expense is necessary to secure such reliability. Reliability is secured through the development of an inexhaustible source or by supplementing a precarious source with adequate reservoirs; by the construction of permanent and substantial structures and equipment; and by the duplication of such structures and machinery the breakdown of a single unit of which might be disastrous to the continuous supply of water.

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[1] WEIBEL, S. R., "A Summary of Census Data on Water Treatment Plants in the United States," *Jour. A.W.W.A.*, December, 1942, p. 1831.

Some sources of supply and methods of distribution offer greater safety and reliability than others. The various types of waterworks are enumerated below in the relative order of their respective reliabilities. It is to be noted that a supply requiring purification is not so reliable as one that is sufficiently pure at its source. Aside from the question of purity, the relative order of reliability is

1. A surface or ground-water supply taken from a never-failing source from which the water is led to, and is distributed throughout, the city by gravity.
2. A surface supply requiring impounding reservoirs from which the water is led to, and is distributed throughout, the city by gravity.
3. A never-failing surface supply which requires pumping for distribution.
4. A surface supply requiring impounding reservoirs from which the water must be pumped.
5. A ground-water supply which must be pumped.

The reliability of any one of these types of waterworks can be made practically unquestioned or extremely uncertain, depending on the design, construction, or maintenance of the works for securing or distributing the water. In the third type of waterworks, the construction of a large storage reservoir with several day's capacity, at an elevation sufficient to distribute the water by gravity, will materially increase the reliability of a waterworks requiring pumping equipment.

In rating a public water supply for the purpose of fixing insurance rates, the reliability of the works is taken into account by the National Board of Fire Underwriters. A gravity supply is preferred by them over one requiring pumping unless there is adequate storage under pressure. Under the circumstance "there is no deficiency charged if, with the two largest [pumping] units out of commission, the remaining pumps, in connection with storage, will furnish the maximum consumption plus fire flow for a 10-hr. period."

The reliability of a water supply is the most important consideration in its selection, particularly for large cities. Other important factors to be considered are quality of water and economy of construction. An impure water may be purified, and the cost of construction should be as low as is consistent with reliability and purity.

5. Governmental Regulation. Public waterworks enterprises almost invariably enjoy a monopoly within the region supplied with water, and hence, whether publicly or privately owned, are subject to governmental regulation.[1] This control is exercised by public utility commissions and commerce commissions, whose jurisdiction extends to equipment, capacities, rates, and financial policies; by water commissions whose rulings may affect the availability of the sources of supply; by health departments who deal with questions of quantity and quality; and by other governmental and quasi-governmental organizations, such as the National Board of Fire Underwriters, who exert their influence through the regulation of fire insurance rates based upon the waterworks equipment.

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[1] Crawford, W. E., and F. G., "Public Utility Regulation," Harper & Brothers, New York, 1933; Beck, J. A., "Regulation of Municipal Plants by Utility Commissions," *Jour. A.W.W.A.*, October, 1935, p. 1323.

6. Legal Rights and Liabilities. The supplying of water is a legally recognized governmental activity,[1] and as such the waterworks enterprise possesses the right of eminent domain[2] in the condemnation of private property for public use.[3] The right to sell water is not, however, a governmental prerogative. Any person may sell water subject only to governmental regulation.[4] Balancing the extensive rights and powers possessed by a waterworks enterprise are its legal liabilities. A waterworks can be sued successfully for damages resulting from the supplying of impure water or from neglect that has injured the consumer through the quantity or quality of the water supplied or through negligence in the operation of the enterprise that has caused personal or property damage.[5] This liability applies alike to publicly and privately owned public water supplies.[6] There is no obligation to serve chemically pure water or water adapted to all commercial purposes.

7. Legal Rights to Sources of Supply. The law and usages governing the right to take water from sources of supply are not uniform in the various states.[7,8] One is impressed with the lack of agreement among authorities respecting many of the points at issue.[9] The following principles govern in different states, and in some states combinations of these principles determine the right to water. The general principles on which rights are based include:

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[1] U.S. Supreme Court Decision, *Jour. A.W.W.A.*, May, 1937, p. 699.

[2] Parker, L. T., "Rights of Municipal Water Works to Appropriate Private Property," *Water Works Eng.*, June 10, 1936, p. 765.

[3] Parker, L. T., "Legal Aspects of Condemnation and Appropriation," *Water Works Eng.*, Nov. 24, 1937, p. 1675.

[4] Parker, L. T., "Laws Regulating Sale of Water," *Water Works Eng.*, July 27, 1932, p. 901.

[5] Parker, L. T., "How to Avoid Legal Pitfalls," *Water Works Eng.*, Jan. 5, 1938, p. 20.

[6] Howard, J H., "Liability for Water-borne Diseases," *Jour. A.W.W.A.*, March, 1945, p. 260.

[7] Conkling, H., "Administrative Control of Underground Water," *Trans. A.S.C.E.*, Vol. 102, p. 753, 1937.

[8] See also "The Appropriation of Water for Beneficial Purposes," *Jour. A.W.W.A.*, July, 1945, p. 601; and "State Water Law in Development of the West," abstracted in *Jour. A.W.W.A.*, January, 1944, p. 102.

[9] See also Parker, L. T., "Municipal Water Supply Rights," *Water Works Eng.*, Sept. 6, 1933, p. 872; and "Water Rights of Riparian Owners," *Water Works Eng.*, July 25, 1934, p. 821.

1. *Priority of Appropriation to Beneficial Use.* This is the principle that "the first in time shall be the first in right," but it is modified in that the use must be beneficial and also by the amount of water available for various preferred purposes.

2. *Ownership Rights.* Ownership rights are acquired through the purchase of land, the rights being specified in the deed thereto. In some cases the rights may include the right to take water from within the boundaries embraced by the deed. This is a very common condition in the location of a well.

3. *Right Is Granted by an Administrative Body.* In some states a just allocation of water rights is attempted through an administrative body acting under legislative authority.

4. *Special Legislation.* The right to acquire water may be secured through special legislative action.

Stevenson and Ryder[1] summarize as follows the guiding principles that have been developed in the appropriation of interstate waters: (1) The doctrine of equitable apportionment controls the division and use of interstate waters. (2) Priority of appropriation creates no superiority of right in interstate waters. (3) The highest use of interstate waters is for drinking and other domestic purposes. (4) Injunctive relief will be granted only upon proof of present and substantial damages. (5) Diversion will be allowed from one watershed to another. (6) States may select sources of water supply provided no substantial damage is done to sister riparian states. (7) Release of compensating water depends on downstream conditions and needs. (8) Judicial allowance of diversion affecting navigation or navigable capacity is conditioned on federal approval.[2]

Johnson[3] divides underground water into two classes: (1) water flowing in defined and known channels, and (2) water passing through the ground below the surface, either without a definite channel or in courses that are unknown. In the conception of the law, waters of the first class are to be dealt with as though they were surface streams. The fundamental principles on which the laws regulating the use of these waters, of the second class, are based is that such percolating subterranean waters are a part of the land itself and belong absolutely to the proprietor within whose territory they are. The land belongs to the owner, whether it be solid rock, porous ground, entirely earthy matter, or part soil and part water. The percolating waters within his territory are as truly his absolute property as are the rocks, ores, or minerals. Consequently, he may take and use such waters as he pleases, although such use may cause damage to his neighbor.

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[1] STEVENSON, W. L., and C. E. RYDER, "New Water Laws in the Making," *Civil Eng*, August, 1931, p. 991.

[2] MERRIMAN, T., "The Law of Interstate Waters," *Jour. New Engl. Water Works Assoc*, Vol. 45, p. 199, 1931; WINSOR, F E., *ibid.*, p. 267.

[3] *U.S. Geol. Survey, Water Supply Paper* 122, 1905.

Conkling[1] states as follows four doctrines of the law on which the use of underground waters in the United States is based: (1) absolute ownership, or the "doctrine of unreasonable use"; (2) ownership to the extent of reasonable use on the land beneath which the water is found, or the "doctrine of reasonable use"; (3) ownership coequal with that of every other owner of land overlying the underground basin, termed the "California doctrine"; and (4) ownership by the state.

The right to take water from any source may be acquired by proper compensation of those enjoying prior rights. This compensation may be directly in money or "in kind." Compensation in kind is usually made by diverting down the stream an amount of water equal to the amount that the water-supply project would otherwise take away from the previous owner or owners lower downstream.

Under any circumstances it may be desirable for the engineer to consult legal advice concerning the rights to any particular source before its use is recommended as a source for a public water supply.

8. The Parts of a Waterworks System. A waterworks includes the structures, equipment, and appurtenances necessary for securing, purifying, and distributing a water supply. Exactly what is, or is not, to be included in a waterworks or a waterworks enterprise may sometimes require legal determination in order to confine the activities of the enterprise within its franchise or other legal limitations.[2]

For purposes of study and design, waterworks may be classified as works for the collection of water, works for the purification of water, works for the pumping of water, and works for the distribution of water. The personnel of a comprehensive waterworks organization includes (1) an engineering organization for the design of structures, machines, and other equipment; (2) an operating force for operating the pumping equipment and control devices on the conduits and on the distribution system; (3) a force for the maintenance of the distribution system; and (4) an office force to care for administrative and financial matters.

Works for the collection of water include wells; galleries; collecting works at springs, lake, and river intakes; impounding reservoirs; aqueducts; and long pipe lines. Works for the purification of water are represented by filtration plants, softening equipment, chlorinating equipment, aerators, and similar devices. Pumps and pumping stations and their equipment represent works for the pumping of water. Works for the distribution of water include the system of underground pipes, distributing and equalizing reservoirs, service connections, and, in some cases, meters. The design, operation, and maintenance of these works is the business of the waterworks engineer and superintendent.

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[1] CONKLING, *loc. cit.*

[2] See also MURDOCH, J. H., JR., "The Legal Limits of a Water Supply System," *Jour. A.W.W.A.*, May, 1943, p. 579.

FIG. 3.—Grounds and tower at Compton Hill Basin, St Louis waterworks. *(Courtesy of John B Dean, Water Commissioner.)*

9. The Business of Supplying Water. A water supply is a public necessity and a public enterprise It is a recognized governmental activity usually conducted as a self-supporting business. Hence, the design, operation, and maintenance of a waterworks should be conducted so that expenditures are economically justified. The customers of a waterworks are the water consumers, and the maintenance of customer good will is essential to successful waterworks management and operation. Customer good will can be created and maintained through education of the public in the problems of the waterworks; by supplying water adequate in both quantity and quality; by the proper type of publicity; by a uniform billing system; and by the beautification and careful maintenance of the buildings, equipment, and grounds belonging to the waterworks.

10. Appearance of Waterworks Properties. Public good will is essential to the ability of a waterworks to attain its principal objective, the provision of an adequate and satisfactory water supply. No waterworks whose structures are dilapidated, whose equipment is uncared for, whose buildings are of lean-to or outhouse appearance, and whose grounds are unkempt can furnish a *satisfactory* water supply. Pleasing architectural style and well-kept property arouse on the part of the public and the water consumers a feeling of confidence and friendship which is measurable in financial returns. The value of waterworks beautification has been proved by experience.

The creation of parks and playgrounds is an effective means for winning public favor. Waterworks parks are maintained in many cities with the provision of playgrounds for sports, picnicking, boating, fishing, and bathing. All can be provided without danger to the quality of the water supplied to the consumers provided proper precautions are taken to prevent the passage of contamination through the water-purification plant.

11. Public Relations.[1] The maintenance of public good will is essential to the continued successful and profitable operation of a public waterworks. If public ill will is earned by a privately owned waterworks, revenge may be obtained by the public through restrictive legislation, increased taxation, loss of franchise, or some other method. If the waterworks is publicly owned, the control by the public is more direct. It may be manifested by a change in the city administration or by a change in the supervisory officials of the waterworks. The American Water Works Association published, in 1947, a Check List of Public Relations Activities valuable in planning and maintaining good public relations. Methods of maintaining public good will represent a special field in waterworks management.

12. Waterworks Management and the Engineer. Among those problems of waterworks management which affect the design, operation, and maintenance of the enterprise may be included: customer relations; policies in the preparation for emergencies; financial policies and procedures; governmental regulation; legal rights and liabilities; methods of measuring water for sale, particularly the use of meters; and the provision of safety measures. Among those important waterworks activities which do not materially affect the engineer may be included methods of accounting and billing; meter reading and its administration; and certain legal responsibilities and obligations such as liability for damages for shutting off the water or for injuries committed on a citizen by a waterworks employee.

Public good will is promoted by the engineer through consideration of esthetics in the design; the provision of an adequate, reliable, and potable supply; and the construction, operation, and maintenance of the works with the least public inconvenience.

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[1] See also Symposium, "Aspects of Public Relations," *Jour. A.W.W.A.*, January, 1947, p. 41.

Emergencies that may be encountered in the operation of a waterworks include fire, flood, tornado, earthquake, labor troubles, cold weather, and drought.[1] Policies to be followed in the preparation for emergencies are determined by the management, but they must all be considered and allowed for by the engineer in design and operation.

13. Starting a Waterworks. Waterworks may be either publicly or privately owned. Private ownership of waterworks is more common in small than in large communities because of the greater ease of raising the necessary finances through private initiative. The origin of a publicly owned waterworks in a community is usually a result of the activities of an influential and interested citizen or group which focus attention on the need therefor. The design of a complete waterworks on a large scale is unusual. The principal work of the waterworks engineer is the design of improvements and extensions to existing systems. Complete works may be designed for small communities installing works for the first time. Under such circumstances an investigation of the possibilities for securing water; purifying it, if necessary; and distributing it, all at a reasonable cost, should be made by a competent engineer whose report may be used as a basis of publicity aimed at winning the support of the citizens, or other responsible body, when the matter is up for consideration and decision. The engineer's report should include a thorough study of all possible sources of supply and of methods of treatment, pumping, and distribution, together with an estimate of the cost of each project. The report should be written in an attractive manner in language easily comprehensible to laymen. An outline of a report is given in Appendix I.

The work of the engineer in connection with a waterworks system can be divided into four periods: development, design, construction, and operation. His traditional duties are confined to design and construction, but he is nonetheless necessary to the complete success of any project in the development and operation periods. The duties of the engineer during the period of development are to gather all the data necessary for his report, to write and present the report, and possibly to make public addresses and to explain to laymen the salient features of the plan or plans presented for consideration.

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[1] See also VANCE, L. S., "Emergencies May Come When Least Expected," *Water Works Eng.*, May 26, 1937, p. 699; "Notification during an Emergency," *Water Works Eng.*, Aug. 18, 1937, p. 1170, CRENSHAW, J. C., "The 1937 Flood at Huntington, W. Va.," *Jour. A.W.W.A*, Vol 29, p. 1230, 1937; and Symposium, "Organization and Preparation of Water Works to Meet Major Catastrophes," *Jour. A.W.W.A.*, Vol. 29, pp. 1737ff., 1937.

FINANCES

14. Financial Principles. The principles involved in the financing of a waterworks enterprise are identical with those applicable to any other business enterprise. A waterworks is a business enterprise. Money must be raised to finance it. This money may be called the capital to be invested. It is the first item in the total capitalized cost, which can be separated into three items, (1) the first cost; (2) the operating cost, which must be large enough so that the simple interest on it will equal the periodic operational charges; and (3) the depreciation fund. The amount of the capitalized cost should be sufficient to purchase, operate, and renew the plant forever. It can be formulated as

$$CC = C + \frac{O}{r} + C\frac{1}{(1 + r)^n - 1} \qquad (1)$$

where CC = capitalized cost
C = first cost or invested capital
O = periodic (usually annual) operating cost
r = periodic (usually annual) rate of interest that can be earned on invested capital
n = life of the enterprise in interest periods (usually years)

In financing an enterprise the first cost represents the invested capital. The income or earnings of the enterprise must be equal to the product of the capitalized cost and the rate of interest on invested capital. Where a different rate of return is allowed on invested capital this income may be modified. The product of the capitalized cost and the rate of interest are called the annual expense. A comparison of the capitalized costs, or the annual expenses, of two or more enterprises capable of performing the same thing will reveal the most economocial of the available choices.

In financing a plant or enterprise, estimates of cost and of depreciation are based on experience, knowledge of market conditions, and published information. Costs fluctuate so greatly that printed information on prices and rates quickly become obsolete.[1] Depreciation rates are less changeable so that published information on them is of greater practical value.[2] In financing an enterprise intended to be either self-supporting or profitable, it is helpful to consider that an assumed rate of interest must be earned on the capitalized cost. The rates to be charged for the services rendered by the enterprise must be sufficient to return a sum equal to the product of the capitalized cost and the assumed rate of interest.

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[1] BURDICK, C. B., "A Review of the Cost of Waterworks Structures," *Jour. A.W.W.A.*, August, 1944, p. 821.

[2] HOWSON, L. R., "Dollars and Sense of Depreciation," *Jour. A.W.W.A.*, August, 1944, p. 803; and VEATCH, N. T., "Municipal Depreciation Accounting Practices," *Jour. A.W.W.A.*, August, 1944, p. 811.

15. Methods of Raising Money. Money may be raised, or borrowed, to finance a business enterprise, such as a waterworks, by (1) direct taxation, (2) special assessment, (3) the sale of general-obligation tax bonds, (4) the sale of revenue bonds, (5) governmental grant or loan, (6) the sale of bonds secured by mortgages, and (7) the sale of stock in the enterprise. In general, all but the last two methods are used for the financing of publicly owned waterworks. The last two are used in the financing of private enterprises. Most of the waterworks of the country are publicly owned, but whether publicly or privately owned, the financial record of waterworks enterprises is so enviable that waterworks securities are usually attractive to investors and enjoy relatively low rates of interest charges. In general, waterworks obligations secured by waterworks revenues are more attractive than obligations secured by the power to tax because experience has shown that it is easier to collect from revenues than from taxes. Publicly owned waterworks are commonly limited by state law in matters of raising money when the obligation is secured by the taxing power.

The waterworks of the larger cities are universally owned by the public. The business of selling water is usually so profitable that there is an irresistible temptation on the part of some city officials to use waterworks funds for other municipal purposes rather than to apply them to the reduction of water rates.

Whenever financial obligations are undertaken that must be retired at some future date, many state laws, together with good business principles, require the obligations to be retired within a reasonable time, certainly within the life of the physical parts of the enterprise, and usually in not more than 20 years. This financial policy affects the period into the future for which structures and equipment should be designed in order that the present generation may not be too greatly burdened for the benefit of future generations. Publicly owned waterworks are usually limited by state laws in matters of raising money for new works and extensions and rehabilitations of existing works. The laws vary in different states.

16. Direct Taxation. Some states permit a direct tax levy to provide funds for public benefit. Such benefit must be shared by the property owners as a whole. The amount of such taxation is usually limited by constitutional, statute, or charter provisions. It is seldom used as a means for waterworks financing, because the required amount is rarely available under the limitations.

17. Special Assessments. A special assessment is a direct tax levy assessed against property to pay for improvements that ordinarily are required to be a direct benefit to the property itself, and in some cases the improvements must be contiguous with the property. Special assessment bonds, subject to statutory limitations, may be issued to provide immediate funds needed for construction purposes. They are secured by the power of the municipality to make and to collect the assessment. Limitations imposed by statute frequently prevent the use of special assessments for waterworks. The special assessment is rarely used in connection with waterworks financing, and the trend is toward its disuse for other municipal purposes.

18. Bonds. The sale of bonds is the most common method of financing waterworks construction. Waterworks bonds, classified as to security, are, in general, of two types: (1) general bonds, secured by the power of the governmental agency to tax or by the real property of the community; and (2) special lien bonds, payable out of the revenues derived from a utility, sometimes called revenue bonds. The physical properties of the utility serve as security for the bonds. Revenue bonds are, however, sometimes issued without hypothecation of the physical properties of the utility and are not a financial obligation of the issuing municipality.[1] Such bonds, despite their apparent lack of sound security, have proved to be financially attractive where the income of the utility is adequate to retire the bonds.

Special lien bonds may also be secured by direct mortgage on the physical property of the utility, in which case provision is usually made for the operation of the utility by the bondholders in case of default. In case of default a receiver may be appointed who is empowered to adjust rates, if necessary and if permitted by the utility commission, in order to protect the investors' interest. The value of the security behind general bonds may be many times the value of the loan. Special lien bonds are usually 100 per cent mortgages having no margin of security value. This fact may affect this type of bond unfavorably in the matter of interest rates.

Bonds may also be classified as long-term bonds, which provide for payment of the entire indebtedness at maturity, or serial bonds, which provide for periodic payments of portions of the indebtedness until the whole is discharged. Provision for the retirement of long-term bonds is usually made by the establishment of a sinking fund. The issuance of serial bonds avoids the hazards of long-term sinking funds, because the bonds themselves are retired with payments that would be invested in the sinking fund. Each type, however, has its advantages and disadvantages.

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[1] See also *Jour. A.W W.A.*, Vol. 27, p. 1343, 1935; and ROBINSON, S. B , "Financing through Revenue Bonds," *Natl. Inst. of Municipal Law Officers Report*, 1944.

Limitations as to the amount of bonded indebtedness, rate of interest, and life of bonds are usually regulated by statutory or constitutional provisions; indebtedness imposed by statute in some states is $2\frac{1}{2}$ per cent of the value of the taxable property; the interest rate must not be more than 6 per cent; and the greatest length of time that bonds may run is 20 years. In some of the New England states the security is the land itself, rather than the power of taxation. Legal requirements usually provide for the method of advertising and disposing of the issue, the selling price, and provisions for retirement. Refunding is not generally permitted.

There has been a recent tendency to exempt public utility debts from provisions limiting the total bonded indebtedness of a municipality. New York's constitution permits such exemptions, provided the improvement returns a net revenue after making a deduction for amortization.

19. Governmental Grants and Loans. The establishment of the Reconstruction Finance Corporation (R.F.C.) by the United States government in 1932, followed by the Public Works Administration (P.W.A.) in 1933, had far-reaching effects on the financing of publicly owned waterworks systems. These two bureaus provided money available to municipal borrowers who wished to construct public works. A self-liquidating feature of the loans required that, in order to be eligible for federal financing, the project must be "self-supporting and financially solvent and indicate that it could earn the interest and repay the principal within a reasonable time by means other than taxation." Since waterworks systems are usually revenue-providing agencies of municipal government, they are eligible to take advantage of such available funds.

The National Industrial Recovery Act (N.I.R.A.) passed by Congress on June 13, 1933, created a Federal Emergency Agency of Public Works, commonly referred to as the P.W.A. This act empowered the President of the United States through the administration to make grants not exceeding 30 per cent (later increased to 45 per cent) of the cost of labor and materials employed in the construction of the project and to finance the remainder of the cost by reasonably secured loans. Other benefits accrued to waterworks systems through legislation establishing the Civil Works Administration (C.W.A.) and Works Progress Administration (W.P.A.), neither of which is now functioning.

20. Water Districts, Sanitary Districts, and Miscellaneous. Some states permit the organization of quasi-municipal corporations for the purpose of providing needed construction or improvement of waterworks, sewerage systems, etc. These are particularly applicable in cases where two or more neighboring municipalities find it advantageous to operate under a single system. Such a plan often permits the issuance of bonds, to the full legal limit, based on assessed valuation, without regard to any existing bonded debt or overlapping municipal corporations.

The situation often arises in which a municipality finds itself unable to finance, by any legal means provided for the contracting of a public debt, either a new installation or needed improvements or extensions to an existing waterworks. The following methods have been used successfully under such circumstances:

1. A holding company is formed which is capitalized for an amount sufficient to cover the entire cost of the needed installation or improvement. The stock of the company is purchased by private individuals, banks, and commercial establishments interested in the civic welfare. The net earnings from the sale of water are used to purchase the stock gradually and to pay a fair interest on the remaining outstanding stock. Thus, the municipality eventually becomes the sole owner of the holding company.

2. Certain private corporations will agree to finance a waterworks installation or improvement under what is known as a "lease form" of contract. In other words, the corporation agrees to build the necessary works and to lease them to the city. The latter agrees to make periodic payments of an amount such that at the end of a term of years the cost, profit, and interest charges will be returned to the financing corporation, and the city becomes the owner of the plant.

21. Financing Extensions.[1] Established waterworks may be expected to extend their services, either by supplying water to suburbs owning their own distribution systems, or by extending the pipes of the distribution system into newly developed territory. The amount to be charged for the supplying of water is a matter of operational costs and usually does not involve new capital charges directly. The extension of mains, however, does represent a capital charge, and the costs must be met as other capital charges are met. The extension should not be undertaken unless there is assurance that the income therefrom will pay the interest on the capital charges of the extension. It must not be overlooked that extensions of water service involve an increase of the load on the entire enterprise. Sooner or later a relatively large capital investment must be made to meet the increased water demand. Each new extension must bear its share of this capital charge. Jackson states[2] that for every dollar invested in distribution mains $0.40 to $2.57 may be invested in other facilities, the average falling a little below $2.

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[1] See also JACKSON, L. A., "Survey of Main Extension Policies," *Jour. A.W W.A.*, August, 1941, p 1357, with references.

[2] *Ibid*, p. 1360.

Sources of revenue for extensions are, of course, the same as for other enterprises. In addition, since the waterworks is now a going concern, current income may be used for financing, or money may be raised through charges made to those about to be benefitted by the extension. Bases for these charges include (1) population served, (2) acreage served, (3) length of frontage benefitted, (4) number of services or meters installed, and (5) prospective use or sale of water. In some cities, where the number of customers, or potential customers, along the extended lines gives assurance of adequate water sales, the cost of the extension is paid for from current income, or other source, and no initial charge is made to the consumer. In other cities, the initial payment advanced by benificiaries is refunded when the sale of water exceeds a predetermined amount sufficient to pay for the extension.

22. Free Water. In many cities the waterworks receives no income for water supplies for public purposes such as fire fighting, schools, or the city building. The custom of supplying "free" water for public use is misleading because the the burden is placed on the water consumer who must carry it in the place of the taxpayer. It would seem more just to place the burden on those who enjoy its benefits.

23. Fire-hydrant Service. Most publicly owned waterworks are not paid directly for fire-hydrant service or for the water used by the fire department. Rates for other services supplied to consumers must be raised to pay for fire protection. However, in some publicly owned waterworks, and in most privately owned waterworks, a direct annual charge is made for each fire hydrant on the distribution system, and frequently the number of fire hydrants per mile of distribution-system piping is specified.

24. Water Rates.[1] The rates to be charged for water sold to a consumer are based on a variety of considerations, such as the cost of the water determined by a valuation of the capital value of the plant and the cost of operation, the value of the fire protection supplied, and the depreciation of the waterworks.[2] Other factors that may be considered include (1) the rate at which water is consumed, (2) the pressure required to supply the demand, (3) the distance of the consumer from the pumping station, (4) the cost of water purification, (5) the percentage of leakage and "lost" water, and (6) the reserve capacity required. That condition on which almost all water bills are based is the amount of water consumed, however inaccurate the measurement of that amount may be.

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[1] See also LEARNED, A. P., "Equitable Schedules of Water Rates," *Jour. A.W.W.A.*, July, 1947, p. 666.

[2] See also NICHOLS, M. C., "Factors in Making Rates," *Jour. A.W.W.A.*, March, 1936, p. 379; and VEATCH, N. T., JR., "Water Works Rate Making," *Jour. A.W.W.A.*, March, 1933, p 343.

Methods in use for estimating the quantity of water consumed for the purpose of fixing the charge paid by the consumer are gaged by the number of fixtures, faucets, or consumers on the service; the number of rooms in the house; the floor area of the building; and meters. Since the quantity of water supplied should be directly proportional to the cost of its production, the correct measurement of this quantity by means of an accurate meter is logically the most equitable method of establishing the charge to be paid by the consumer.[1]

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[1] See also SCHWADA, J. P., and E. F. TANGHE, "Some Factors Entering into the Determination of Water Rates," *Jour. A.W.W.A.*, December, 1943, p. 1528.

CHAPTER II

HYDRAULICS

25. Problems Involved. Hydraulics may be considered to be a general study of the principles that control the behavior of liquids, particularly water, when at rest or in motion. The design, maintenance, and operation of waterworks involve problems in the collection, pumping, storage, distribution, and purification of water that require a broad knowledge of hydraulics. Fundamentals of hydraulics applicable to the solution of problems commonly met in waterworks practice are discussed in this chapter. Other principles, applicable to special problems, are discussed in other chapters. For a complete discussion of hydraulic principles; textbooks on hydraulics should be consulted.

Hydrostatics, a branch of hydraulics, involves problems of water at rest. Such problems include intensity of pressure, total pressure, and center of pressure. Hydrodynamics deals with water in motion and includes problems in the flow of water in pipes, conduits, and distribution systems; pumps; water hammer; and others.

26. Intensity of Pressure. The intensity of pressure exerted on a surface submerged in water can be expressed as

$$p = wh \tag{1}$$

where p = intensity of the pressure
w = unit weight of water
h = distance from the water surface to the area on which the pressure is exerted

The expression is correct under all conditions except where there is a component of velocity normal to the submerged surface.

Since the weight of water is approximately 62.5 lb. per cu. ft., the intensity of pressure, in pounds per square inch (p.s.i.), at the foot of a column of water 100 ft. high can be computed as

$$p = wh = \frac{62.5 \times 100}{144} = 43.4 \text{ p.s.i.}$$

The height of a column of water above a point where the pressure is 100 p.s.i., can be computed as

$$h = \frac{p}{w} = \frac{100 \times 144}{62.5} = 231 \text{ ft.}$$

27. Total Pressure on Plane Surface. The total pressure normal to a submerged plane surface can be expressed as

$$P = w\bar{h}A \tag{2}$$

where P = total pressure
w = unit weight of water
$\bar{h}$ = distance from the water surface to the centroid of the submerged surface
A = area of the submerged surface

The direction of action of the resultant of the total pressure is normal to the plane of surface, and its point of application is at the center of pressure.

28. Components of Total Pressure on a Curved Surface. The direction of action of pressure against a surface is normal to the surface. Hence, the summation of the pressures in various directions, representing the total pressure, is of little more than academic interest. The magnitude of the resultant of all the components of the total pressure in any direction is, however, of practical interest. It can be expressed as

$$P = \int wH \cos \alpha \, dA \tag{3}$$

where P = total pressure
H = depth of submergence of a differential area of the submerged surface
α = angle between a line parallel to the direction of P and a line normal to dA
A = area

Both H and α are variables, and unless the relation between them is known the expression cannot be integrated. Where H or α is constant, the expression becomes

$$P = wH \int \cos \alpha \, dA \qquad \text{or} \qquad w \cos \alpha \int h \, dA - w\bar{H}A'$$

where A' is the area of projection of the curved surface in a direction parallel to the resultant P on to a plane surface normal to this direction, H is approximately constant where the depth of submergence is great, and α is constant where the submerged surface is a plane. Where A' is a vertical plane

$$\int H \cos \alpha \, dA = \int H \, dA' = \bar{H}A' \qquad \text{and} \qquad P = w\bar{H}A'$$

Hence, the pressure in a horizontal direction against any surface is equal to the horizontal pressure against the horizontal projection of that surface against a vertical plane.

29. Center of Pressure. In Fig. 4 the distance to the center of pressure of the submerged plane surface can be expressed as

$$Z_s = \frac{I_s}{S_s} \tag{4}$$

where Z_s = distance, measured in the plane of the submerged surface, from water surface to center of pressure
I_s = moment of inertia of submerged surface around the axis formed by intersection of submerged surface with water surface
S_s = gravity moment of submerged surface around the same axis

The distance y_1 of the center of pressure of a plane surface from any axis lying in the plane of the submerged surface and perpendicular to the above-described axis is

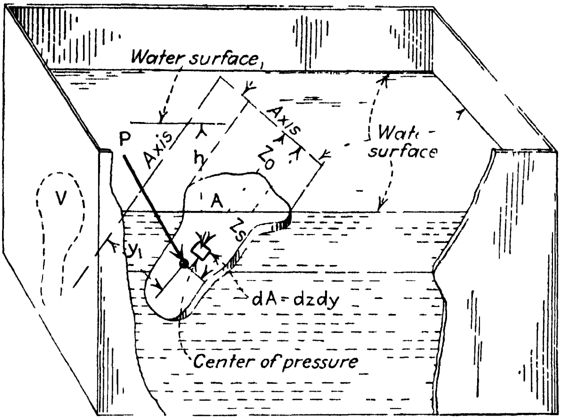


FIG. 4.—Diagram showing the location of the center of pressure of a submerged surface.

$$y = \frac{\int zy\,dA}{\int z\,dA} \tag{5}$$

where y = horizontal distance from axis to differential area $dA = dz\,dy$
z = distance from the axis in surface of liquid to differential area dA

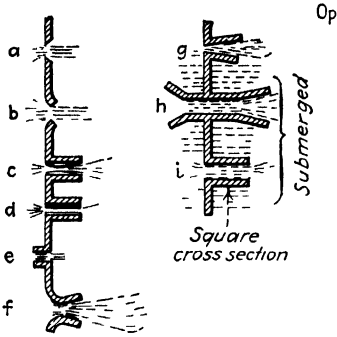
Where the submerged surface is nonplanar, the distance to the center of pressure can be expressed as

$$h = \frac{\int h^2\,dA}{\int h\,dA} \tag{6}$$

and the distance y is as shown in expression (5). An integrable relation between H and A must exist to permit the solution of the problem.

Approximate solutions may be reached by assuming the curved surface to be composed of a series of plane surfaces.

30. Flow through Orifices and Nozzles. Knowledge of the rate of flow through an orifice may be applied to the determination of the rate of discharge through a valve, the rate of lowering of water in a storage tank, or the measurement of the rate of flow in a pump or in a well test.



Coefficients of Discharge

Opening	Coef.	Remarks
a	0.60-0.65	Standard
b	0.98±	
c	0.82±	
d	0.60-0.65	
e	0.53±	
f	0.870 to 0.994	Coef. depends on head, shape of orifice and flare of tube
g		
h	0.785-2.35	Ditto
i	0.599-0.802	

FIG. 5.—Coefficients of discharge for orifices and short tubes. All are circular in cross section and discharge into the atmosphere except as noted (See also *Univ. Ill Expt Sta Bulls.* 96 and 100. *Eng. News*, Vol. 76, pp 520 and 826, *Eng. News-Record*, Vol. 86, p. 24.)

The rate of flow through an orifice or nozzle can be expressed as

$$Q = C_1AC_2\sqrt{2gh} = K_0A\sqrt{2gh} \tag{7}$$

where Q = rate of flow through orifice
C_1 = coefficient of area of reduction
C_2 = coefficient of velocity
K_0 = coefficient of discharge
g = acceleration due to gravity
h = head on the center of orifice, or centroid of orifice opening
A = area of orifice

Coefficients for various orifices and nozzles, set far enough from the walls of the tank or container to avoid interference therefrom, are shown in Fig. 5. Values of K_0 for orifices have been found to vary from 0.58 to 0.71. Values of K_0 for smooth conical nozzles vary between 0.95 and 0.99.

Where a large orifice is discharging under a low head, *i.e.*, when the height of the opening is large compared with the depth of water over it, modification of the formula is necessary. For a rectangular orifice, the formula becomes

$$Q = mL(H_1^{3/2} - H_2^{3/2}) \tag{8}$$

where Q = rate of discharge through orifice
L = width of orifice
H_1 = head on bottom of orifice
H_2 = head on top of orifice
m = a coefficient that depends on the shape and condition of installation of orifice

The order of magnitude of m and some conditions affecting it were illustrated in a test on the crest gates of the Wilson Dam.[1] The spillway has 58 gates with clear openings of 58 ft. each. With one gate open to different heights, and adjacent gates closed, it was found that the value of m varied between 3.23 for small openings to 3.83 for full opening. Different openings for one gate, with adjacent gates fully open, gave values of m from 3.33 for small heads to 4.17 for full opening.

31. Discharge under Falling Head. Problems involving the time required to lower the elevation of the water surface in a reservoir or tank by opening a submerged gate or the distance the water surface will fall in a given time under similar conditions are illustrative of the principle of discharge under falling head. The relation between time and distance of fall can be expressed as

$$t = \frac{A}{ca\sqrt{2g}} \int_{h}^{h_1} \frac{dh}{\sqrt{h}} \qquad (9)$$

where t = time
A = surface area of reservoir or tank
c = coefficient of discharge of orifice or gate
a = area of orifice or gate
h_1 = head on orifice at start of time t
h_2 = head on orifice at end of time t
h = head on orifice at any time

If A is a function of h, as sometimes happens, then

$$t = \frac{1}{ca\sqrt{2g}} \int_{h_2}^{h_1} f(h) \frac{dh}{\sqrt{h}} \qquad (10)$$

32. Discharge over Measuring Weirs. A measuring weir is distinguished from a weir in that the former is a device for measuring the rate of flow of water, formed by a notch or crest in the top edge of a vertical plate or diaphragm over which the water to be measured falls, and the latter is a dam across a stream used for diverting or measuring the rate of flow. Measuring weirs may be "standard," broad crested, sharp, curved, or flat; suppressed or contracted; rectangular; V-notch; trapezoidal; semicircular; free falling; or submerged.

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[1] Puls, L. G., "Spillway Discharge Capacity, Wilson Dam," *Trans. A.S.C.E.*, Vol. 95, p. 316, 1931.

A standard weir consists of a sharp horizontal edge of a flat plate over which water flows in such a manner that there is air beneath the nappe of the water falling over the weir. The rate of flow over such a weir, without end contractions, is

$$Q = clh^{3/2} = 3.33lh^{3/2} \tag{11}$$

where c = a coefficient, usually 3.33 for a standard crest
l = length of weir crest, ft.
h = head on weir, ft.
Q = rate of discharge, c.f.s.

A weir of this type, without end contractions, is known as a suppressed weir. The formula is subject to modifications, depending on contractions, aeration, velocity of approach, shape of crest, submergence, and possibly other factors.

Where there are end contractions the formula for a standard weir becomes

$$Q = 3.33\left(l - \frac{nh}{10}\right)h^{3/2} \tag{12}$$

where n = number of contractions, either 1 or 2

With a velocity of approach, the flow over a suppressed weir is

$$Q = 3.33l\left[\left(h + \frac{V^2}{2g}\right)^{3/2} - \left(\frac{V^2}{2g}\right)^{3/2}\right] \tag{13}$$

where V = velocity of approach, f.p.s.

With a velocity of approach, the flow over a contracted weir is

$$Q = 3.33\left(l - \frac{nh}{10}\right)\left[\left(h + \frac{V^2}{2g}\right)^{3/2} - \left(\frac{V^2}{2g}\right)^{3/2}\right] \tag{14}$$

The discharge Q_s over a submerged weir may be taken as a function of the discharge over a free-discharge weir Q_f. Using the formula $Q_s = KQ_f$, values of K determined through model tests by the U.S. Deep Waterways Commission[1] are given in Table 2.

TABLE 2.—COEFFICIENTS FOR SUBMERGED WEIRS

Hd/H*	K	Hd/H	K
0	1.000	0.5	0.937
0.1	0.991	0.6	0.907
0.2	0.983	0.7	0.856
0.3	0.972	0.8	0.778
0.4	0.956	0.9	0.620

* H = upstream head on weir, H_d = downstream head on weir.

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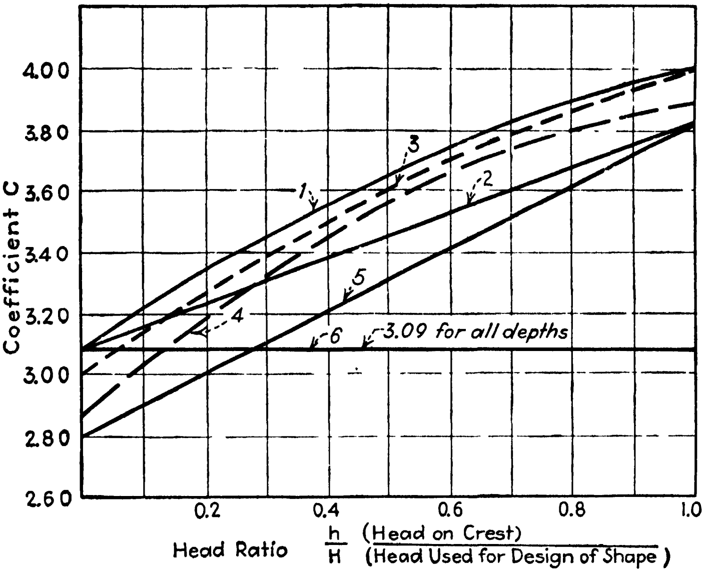
[1] *U.S. Geol. Survey, Water Supply and Irrigation Paper* 200, 1907.

For a triangular-notch weir

$$Q = C \tan \alpha h^{5/2} \tag{15}$$

where α = angle of sides of notch with the vertical
h = depth of water from bottom of notch to the surface in the approach channel
C = a coefficient
$\alpha = 45°$
$Q = 2.5h^{5/2}$

Tests[1] indicate that measurements with heads below 0.3 ft. on triangular-notch weirs are not dependable.



1, *C* for free-flow straight ogee crest.
2, *C* for drum gate crests.
3, *C* for three or more openings of controlled ogee crest.
4, *C* for one opening of several controlled ogee crests.
5, *C* for ring-shaped or shaft spillway crest.
6, *C* for open-channel spillway no crest (control section).

When submerged apply coefficients in Table 2, p. 26. Effect of end contractions included in *C* or assumed negligible.

FIG. 6.—Approximate values of *C* for spillway crests. (*Courtesy of Bureau of Reclamation.*)

33. Broad-crested Weirs. Where the shape of the weir is not sharp-edged, and contractions and velocity of approach can be neglected, a condition that exists over the spillways of many dams, then the flow over the spillway can be expressed as

$$Q = Clh^{3/2} \tag{16}$$

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[1] See also BLAISDELL, F. W., "Discharge of V-notch Weirs at Low Heads," *Civil Eng.*, August, 1939, p. 495.

Values of the coefficient C are affected by the shape of the crest and the magnitude of h. Some values of C are given in Fig. 6.

A broad-crested weir is defined as "an overflow structure on which the nappe is supported for an appreciable length; a weir with a significant dimension in the direction of the stream."[1] The formula $Q = Clh^{3/2}$ is often used to determine the discharge of a broad-crested weir. The value of the coefficient C varies, with the head and shape, from 2.5 to 3.1.[2] The subject is too broad to be adequately covered in the space available without danger of creating erroneous impressions. The reader is therefore advised to consult standard textbooks and references for information.

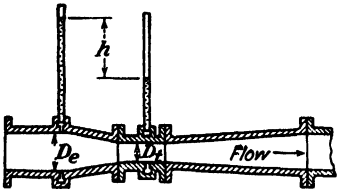

FIG. 7.— Venturi meter.

34. Ogee Spillway Crests. Masonry crests are now usually designed to fit, nearly, the lower nappe which would be formed by a jet flowing over a sharp-crested weir at a head corresponding to the design head for the spillway. This practice tends to cause the pressure at the crest to approach zero or atmospheric and thus make more of the head available for producing velocity. Assuming that the rate of flow Q over a sharp-crested weir having a length L and operating under a head h is $Q = 3.33Lh^{3/2}$, and also assuming that the rise of the lower nappe is $0.11h$, making $H = 0.89h$, the formula of flow would become $Q = 4.10LH^{3/2}$ for a perfectly shaped rounded crest when discharging at its designed head. Coefficients for rounded crests usually lie between 3.0 or less for low heads and 4.0 for well-shaped crests at designed heads.

35. Velocity Meters. The venturi meter, shown in Fig. 7 was invented by Clemens Herschel.[3] The rate of flow through the meter can be expressed as

$$Q = C \frac{\pi D_e^2 D_t^2}{4(D_e^4 - D_t^4)} \sqrt{2gh} \qquad (17)$$

where D_e and D_t = diameters at the entrance and at the throat, respectively

h = difference between pressure at entrance and pressure at throat

C = a coefficient

For most meters the value of C is unity. Formula (17) can be derived directly from an algebraic expression of Bernoulli's theorem.

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[1] "Manual of Engineering Practice No 11," A.S C E , 1936

[2] WOODBURN, J. G., "Tests of Broad-crested Weirs," *Trans. A S.C.E.*, Vol. 96, p. 387, 1932; *U.S. Geol. Survey, Water Supply Paper* 200, 1907.

[3] HERSCHEL, C., "The Venturi Meter," *Trans. A.S.C.E.*, Vol. 17, p. 228, 1887.

Venturi meters are extensively used for the metering of water in pumping stations, treatment plants, and other locations where high rates of flow are to be measured. Indicating and recording mechanisms are available to indicate the momentary rate of flow or to integrate the cumulative volume of flow through the meter.[1]

Elbows in pipe lines may be used to measure the quantity of water flowing in the line.[2] The difference in pressure head h_p between the inner and the outer edge of the bend is measured by means of a differential gauge. The formula

$$V = Kh_p^{1/2} \tag{18}$$

in which K equals a constant, may be used to determine the mean velocity V around the bend.

The value of K must be determined by calibration for accurate results. It varies with the ratio of diameter of pipe to radius of bend, approach conditions, and type of pipe. Tests by Lansford[3] at the University of Illinois indicate that in English units K varies from about 7.0 for 1-in. pipe to about 5.0 for 24-in. pipe. The 1-in. pipe bends tested were in ordinary steel pipe with short-radius screw elbows. The 24-in. pipe bends were in spiral-riveted steel pipe with segmented elbows, having a ratio of diameter of pipe to radius of bend equal to about 0.8.

36. Other Methods of Measuring Rates of Flow. Among many methods used for the measurement of high rates of flow may be included the Pitot tube and the current meter. Both show velocity of flow, the Pitot tube being more commonly used in closed conduits and the current meter in open channels. The rate of flow through the conduit is the product of the measured velocity and the cross-sectional area of the stream at the point where the velocity was measured.

37. Measuring Flumes. Open flumes with a constricted throat section, sometimes called venturi flumes, can be used for measuring the rate of flow of water. A true venturi flume requires the measurement of the depth of flow in the flume and at the throat. A Parshall flume is a special type of venturi flume, see Fig. 8. The flow through such a flume can be expressed as

$$Q = 4.1WH^{3/2} \tag{19}$$

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[1] See also REGESTER, R. T., "Main-line Metering of Water, Sewage and Sludge," *Water Works & Sewerage*, June, 1941, p. R-31.

[2] JACOBS and SOOY, "New Method of Water Measurement by Use of Elbows in a Pipe Line," *Jour. Elec. Power Gas*, July 22, 1911; YARNELL and NAGLER, "Flow of Water in Bends in Pipes," *Trans. A.S.C.E.*, Vol. 100, p. 1018, 1935; LANSFORD, W. M., "Discussion," *ibid.*, p. 1036.

[3] LANSFORD, W. M., "The Use of an Elbow in a Pipe for Determining the Rate of Flow," *Univ. Ill. Eng. Expt. Sta. Bull.* 289, 1936.

where W = width at throat of flume
H = depth, shown in Fig. 8, at point where depth observations are made

The use of venturi flumes is not common in waterworks practice because public water supplies are not commonly conveyed in open channels.

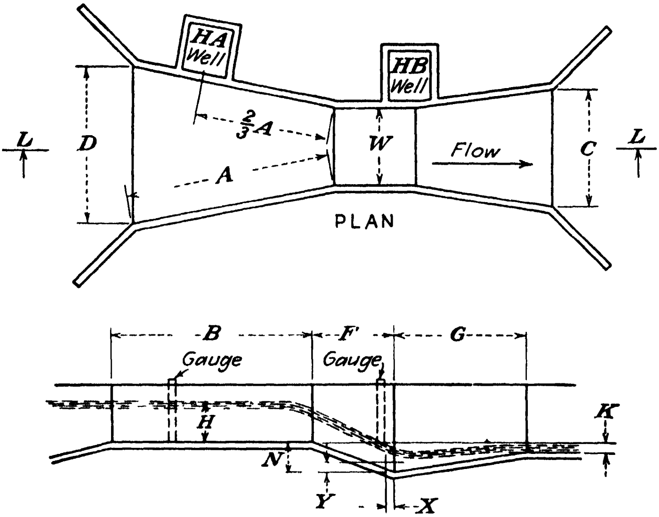

FIG. 8.– A Parshall flume.

38. Salt-velocity Method of Measurement. Allen and Taylor[1] have introduced a method for the measurement of the rate of flow of water. In this method salt solution is introduced into a conduit and the time of passage of the solution between pairs of electrodes placed in the conduit is observed. The rate of discharge through a conduit flowing full is equal to the volume of conduit between electrodes divided by the time of passage of the salt solution. Mason[2] points out some of the dangers of using this method without full knowledge of the factors involved. Among the conditions affecting the accuracy of the method may be included uniformity of cross section between electrodes and the avoidance of large-scale eddies between electrodes.

39. Flow in Channels. It is commonly assumed that where water is flowing in a pipe the energy at any point in the flowing stream is equal to the energy at any other point downstream plus the loss of energy due to friction in flowing between the points. This principle, generally known as Bernoulli's theorem, can be expressed as

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[1] See ALLEN, C. M., and E. A. TAYLOR, "The Salt-velocity Method of Water Measurement," *Trans. A.S.C.E.*, Vol. 45, p. 285, 1923.

[2] MASON, M. M., "Contribution to a Study of the Allen Salt-velocity Method of Water Measurement," *Jour. Boston Soc. Civil Eng.*, Vol. 27, p. 207, 1940.

$$z_a + h_a + \frac{V_a^2}{2g} = z_b + h_b + \frac{V_b^2}{2g} + h_f \tag{20}$$

where V_a and V_b are velocities of flow at A and B, respectively, and other nomenclature is illustrated in Fig. 9.

It can be shown by mathematical analysis that if the velocity of flow is constant between any two points the head lost h_f, for turbulent flow, varies as the velocity squared. The relationship can be expressed as

$$h_f = kV^2 \tag{21}$$

The value of k depends on conditions such as the roughness, shape, and dimensions of the channel; the velocity of flow; the characteristics of the fluid; and the dimensions used. Values of k are determined empirically.

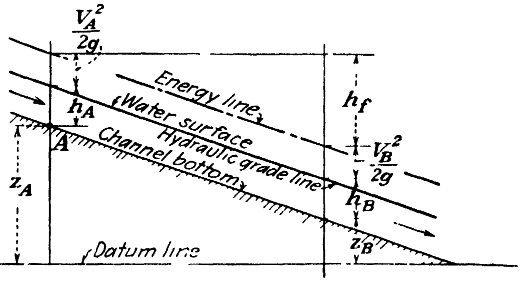

FIG. 9.—Bernoulli's theorem applied to open-channel flow.

40. Flow Formulas. The numerous formulas available for solution of problems in the flow of water, with coefficients based on incompatible qualitative descriptions of conduit materials, add confusion and uncertainty to the use of any of them. Some formulas are shown in Table 3. The inclusion of Reynolds number[1] and Froude's number[2] in recently published formulas makes possible their use for fluids other than water, a refinement not widely applicable in routine waterworks practice. A real value of these numbers, particularly Reynolds number, is their applicability in the prediction of the performance of large conduits from experiments in small ones. Practice is generally focused on the use of the Hazen and Williams formula.[3] Scobey's[4] statement in presenting his

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[1] See REYNOLDS, OSBORN, *Phil. Trans. Roy. Soc. (London)*, 1882, 1885.

[2] See FROUDE, W., *Trans. Inst. Naval Arch.*, Vol. 11, p. 80, 1870.

[3] WILLIAMS, G. S., and ALLEN HAZEN, "Hydraulic Tables," John Wiley & Sons, Inc., New York, 1st ed., 1905, with subsequent editions. *Jour. New Engl. Water Works Assoc.*, September, 1935.

[4] SCOBEY, F. C., "The Flow of Water in Riveted Steel and Analogous Pipes," *U.S. Dept. Agr., Tech. Bull.* 150, p. 18, 1930.

TABLE 3.—FORMULAS FOR THE FLOW OF WATER

Name	Formula
Weisbach or Darcy[1]......	$h = f\dfrac{l}{D}\dfrac{V^2}{2g}$
Chézy[2]..................	$V = C\sqrt{RS}$
Kutter[3].................	$V = \left[\dfrac{\dfrac{1.81}{n} + 41.67 + \dfrac{0.0028}{S}}{1 + \dfrac{n}{\sqrt{R}}\left(41.67 + \dfrac{0.0028}{S}\right)}\right]\sqrt{RS}$
Manning[4]...............	$V = \dfrac{1.486}{n} R^{2/3}S^{1/2}$
Rhodes[5].................	$V = 17.5R_n{}^{0.085}F_n{}^{-0.02}D^{0.5}S^{0.5}$
Capen[6]..................	$V = F_mD^{0.625}H_1{}^{0.5}$
Bazin[7].................	$V = \dfrac{87}{0.552 + \dfrac{m}{\sqrt{R}}}\sqrt{RS}$
Hazen and Williams[8]......	$V = CR^{0.63}S^{0.54}(0.001^{-0.04})$
Scobey[9]................	$V = CH_1{}^{0.5}(12D)^{0.625}$
Flamant[10]	$V = CR^{5/7}S^{4/7}$

[1] "*Recherches experimentales relatives au mouvement de l'eau dans les tuyaux*," Paris, 1857.

[2] Algebraically the same as Weisbach or Darcy.

[3] GANGUILLET, E., and W. R. KUTTER, "A General Formula for Flow of Water in Rivers and Other Channels," 2d ed., translated by J. C. Trautwine, John Wiley & Sons, Inc., New York, 1907.

[4] MANNING, ROBERT, *Trans. Inst. Civil Eng. of Ireland*, Vol. 20, 1890.

[5] *Eng. News-Record*, Aug. 15, 1940, p. 45.

[6] *Jour. A.W.W.A.*, Vol. 33, p. 1, 1941.

[7] BAZIN, M., *Annales ponts et chaussées*, October, 1888.

[8] "Hydraulic Tables," 1 ed., John Wiley & Sons, Inc., New York, 1905, and subsequent editions.

[9] *U.S. Dept. Agr., Bull.* 376, 1916; *Bull.* 852, 1924 *Bull.* 150, 1930.

[10] FLAMANT, A., *Annales ponts et chaussées*, September, 1892.

Nomenclature:

C = a coefficient, depending on roughness of pipe walls
D = diameter of pipe, ft.
f = friction factor in Weisbach-Darcy formula. Value depends on roughness of pipe walls
F_m = friction factor in Capen's formula
F_n = Froude's number = $V/\sqrt{Dg}$
g = acceleration due to gravity, ft. per sec.^{2}
H = head loss, or friction loss, ft. for length of 1 ft.
H_1 = head loss, or friction loss, ft. per 1,000 ft.
k = absolute roughness, or height of surface roughness, or some linear dimension representing height, form, and distribution of such projections
l = length of pipe, ft.
m = friction factor, or coefficient of roughness, in Bazin's formula
n = friction factor, or coefficient of roughness, in Kutter's and in Manning's formulas
R = hydraulic radius of channel, ft.
R_n = Reynolds number = VD/ν = $VD\rho/\mu$
S = slope of hydraulic grade line in uniform flow = H/l
V = velocity of flow, f.p.s.
ν = kinematic viscosity
ρ = density
μ = relative viscosity

formula is significant in this connection. He states: "Engineers not wishing to adopt the new formula . . . should not digress farther than to use the Williams-Hazen formula, which conforms with reasonable closeness to that of the writer for usual sizes and velocities."

TABLE 4.—VALUES OF *n* IN KUTTER'S AND IN MANNING'S FORMULAS

Material	*n*
Well-planed timber	0.009
Neat or very smooth cement	0.010
Unplaned timber, best concrete	0.012
Smooth masonry or brickwork or ordinary concrete	0.015
Rubble masonry or rough brickwork	0.017
Smooth earth	0.020–0.030
Rough grassy channels	0.030–0.050

Comparisons and discussions of the various formulas are frequently published. Among the more comprehensive articles may be included those of Capen,[1] with an extensive bibliography and with subsequent discussion,[2] and Hinds.[3]

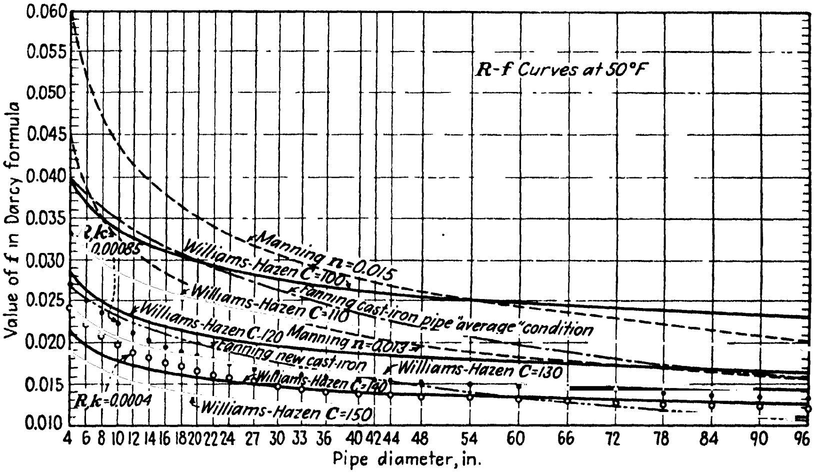

FIG. 10.—Comparative curves for *f* values at a mean velocity of 3 f.p.s. (*From Leo Hudson, "Analysis of Pipe Flow Formulas in Terms of the Darcy Function f," Jour. A.W.W.A., June,* 1947, *p.* 570.)

The Weisbach-Darcy formula

$$h = f\frac{l}{D}\frac{V^2}{2g}$$

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[1] CAPEN, C. H., "Trends in Coefficient of Large Pressure Pipes," *Jour. A.W.W.A.*, January, 1941, p. 1.

[2] *Jour. A.W.W.A.*, April, 1942, p. 523.

[3] HINDS, JULIAN, "Comparison of Formulas for Pipe Flow," *Jour. A.W.W.A.*, November, 1946, p. 1226.

may be considered as rational. It can be developed mathematically from Bernoulli's theorem, a mathematical expression of the principle of the conservation of energy. The trend in practice is to use this formula, in connection with Reynolds number, in problems involving fluids other than water and for the extension of experimental data beyond the limits of experimental observations. Values of f are shown in Fig. 10.[1] Values of f in a large tunnel are shown by Hicox[2] to be affected by Reynolds number. Relative roughnesses are shown in Fig. 11, and some values of k are given in Table 5.

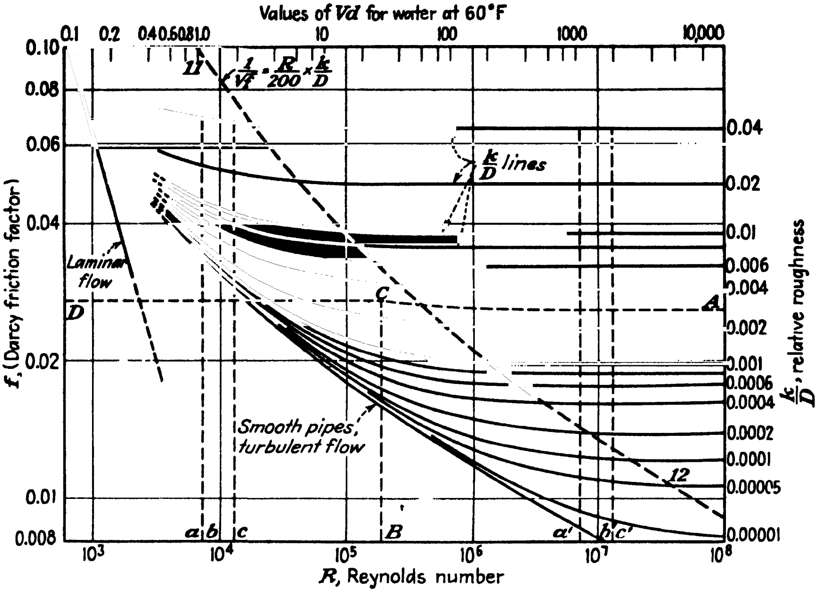

FIG. 11. Solution of Colebrook formula for rough pipe. (*From G. H. Hickox, Proc. A.S.C.E., April*, 1947.)

The Chézy formula is algebraically identical to the Weisbach-Darcy formula. It is usually expressed as $V = C\sqrt{RS}$, in which V is the velocity of flow, C is a constant, R is the hydraulic radius, and S is the slope of the hydraulic grade line in uniform flow.

---

[1] From HUDSON, LEO, "Analysis of Pipe Flow Formulas in Terms of the Darcy Function f," *Jour. A.W.W.A.*, June, 1947, p. 568; see also MOODY, L. F., "Friction Factors for Pipe Flows," *Trans. A.S.M.E.*, Vol. 66, p. 671, 1944; and HINDS, JULIAN, "Comparison of Pipe Flow Formulas," *Jour. A.W.W.A.*, November, 1946, pp. 1226, 1243, 1244, and April, 1945, p. 405.

[2] Hicox, G. H., and others, "Friction Coefficients in a Large Tunnel." *Proc. A.S.C.E.*, April, 1947, p. 451.

TABLE 5.—ABSOLUTE ROUGHNESS k OF VARIOUS MATERIALS

Material	k	Kutter's and Manning's n	Corresponding value of k
Drawn tubing	0.000005	0.0105	0.00015
Commercial steel or wrought iron	0.00015	0.011	0.0005
Asphalted cast iron	0.0004	0.012	0.0002
Galvanized iron	0.0005	0.013	0.005
Wood stave	0.0006–0.003	0.014	0.011
Cast iron	0.00085	0.015	0.02
Concrete	0.001 –0.01	0.016	0.03
Riveted steel	0.003 –0.03		

TABLE 6.—VALUES OF C AND OF K_1 IN HAZENS AND WILLIAMS FORMULA
(For circular pipes running full)

Material	C	K_1
New tar-coated pipes:		
Supply and transmission mains larger than or equal to 16 in.	135	0.35
Distribution mains smaller than 16 in.	125	0.405
New cement-lined pipes:		
Centrifugal method, based on actual diam. where diam. is between 4 and 24 in.	150	0.28
New cast-iron pipe, pit cast	120 130	
New cast-iron pipe, centrifugally cast	125 135	
Cement lining, applied by hand	125 135	
Bitumastic enamel, hand brushed	135–145	
Bitumastic enamel, centrifugally applied	145-155	
Concrete, best workmanship, large diam	145 155	
Ordinary tar-dipped cast-iron, 20 years service in inactive water.	110 125	
Ordinary tar-dipped cast-iron after long service with severe tuberculation	30 40	
Ordinary tar-dipped cast iron; average effect of tuberculation on values of C: new, 135; 5 years old, 120; 10 years old, 110; 15 years old, 105; 20 years old, 95; 30 years old, 85; 40 years old, 80		
New bituminous enamel-lined pipes		
Supply and transmission pipes 16 in. or larger	155	0.27
Smaller than 16 in.	145	0.30
Transite pipe:		
6-in.	140+	0.32
Old tuberculated cast-iron pipe after 20 years of service	100	0.61

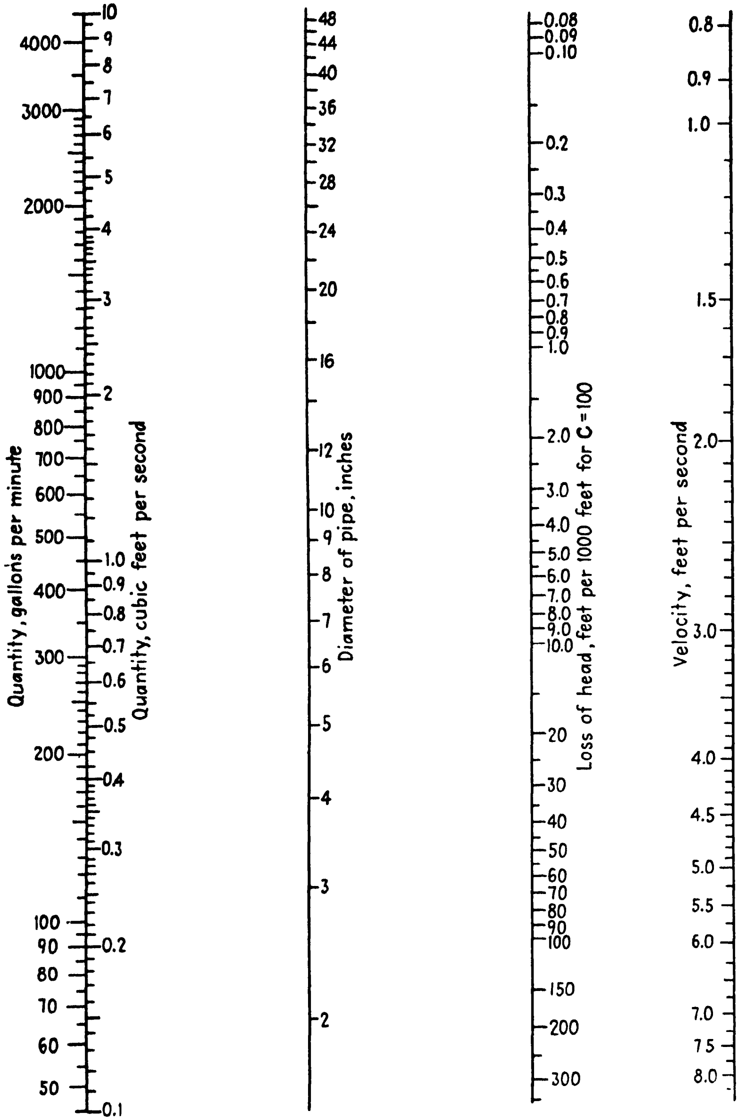

FIG. 12.—A nomographic chart for the solution of problems involving the Hazen and Williams formula. Use of diagram: Place a straightedge on two known points and read at the intersection of the straightedge with the verticals the values of the other two hydraulic elements which satisfy the condition. When only Q and V are given the problem is indeterminate.

The Chézy, Kutter, and Manning formulas are most commonly used in problems involving open-channel flow.

41. Hazen and Williams Formula. The Hazen and Williams formula is the formula most commonly used in the solution of problems in the flow of water in closed conduits. The formula can be expressed as shown in Table 3 or as

$$H = \frac{3.030V^{1.85}}{C^{1.85}D^{1.17}} = K_1\frac{V^{1.85}}{D^{1.17}}$$

The nomenclature used is shown at the bottom of Table 3. Values of C and of K_1 are shown in Table 6.

Solutions of problems involving Hazen and Williams formula have been simplified by the use of slide rules and charts. A nomographic chart is shown in Fig. 12 based on the value of $C = 100$. Engineers sometimes refer, indirectly, to the Hazen and Williams formula when describing the characteristics of a pipe as a "100 pipe" or a "160 pipe," depending on the coefficient of roughness. Since V and Q vary directly with the coefficient C, Fig. 12 can be read directly with $C = 100$. The correct value is equal to the product of the true value of C and the observed value of V, or of Q, respectively, read from the chart, divided by 100.

Example: Find the rate of flow and the velocity of flow in a bituminous-lined pipe 16 in. in diameter, when the head loss is 1 ft. per 1,000 ft. and $C = 160$.

Solution: Reading from Fig. 12, $Q = 1{,}000$ and $V = 1.6$. The corrected value of Q is $1{,}000 \times 160/100 = 1{,}600$ g.p.m., $V = 1.6 \times 160/100 = 2.6$ f.p.s.

The value of S or of H for values of C other than 100 must be computed by multiplying the value read from the chart by $(100/C)^{1.85}$.

Example: Find the head lost, in feet, and the velocity in feet per second, in 2,000 ft. of 16-in., bitumastic-lined pipe when $Q = 1{,}600$ g.p.m., and $C = 160$.

Solution: Reading from the chart, $H = 2.4$ and $V = 2.6$. The head loss in 2,000 ft. is $(2.4 \times 2)(^{100}/_{160})^{1.85} = 2$ ft.

When two of the three terms Q, V, and D are known, the third can be read directly from the chart, since regardless of the value of C,

$$Q = AV = \left[\frac{\pi D^2}{4}\right] V$$

Hence, in the preceding example $V = 2.6$.

42. Effect of Temperature on Pressure and Flow. In general, the effect of change of temperature on pressure and on rate of flow is so slight that it is neglected in practice. The effect of temperature on density and viscosity of water is shown in Table 7. These figures show that the

Table 7.—Density and Viscosity of Water at Different Temperatures

Temp., °F.	Density ρ, lb./cu. ft.	Viscosity μ, lb./ft. sec.	$\rho/\mu = 1/\nu$	Temp., °F.	Density ρ, lb./cu. ft.	Viscosity μ, lb./ft. sec.	$\rho/\mu = 1/\nu$
32	62.42	0.001206	51,754	90	62.12	0.000513	121,091
40	62.42	0.001039	60,082	100	62.00	0.000458	135,371
50	62.41	0.000880	70,918	150	61.20	0.000291	210,309
60	62.37	0.000759	82,173	175	60.69	0.000241	251,825
70	62.30	0.000660	94,393	200	60.14	0.000205	293,365
80	62.22	0.000579	107,611	212	59.84	0.000141	313,314

pressure on a surface submerged in water at 40°F. is 62.42/62.00 times the pressure at 100°F.

The effect of viscosity and density on flow are cared for in Darcy's formula by Reynolds number.

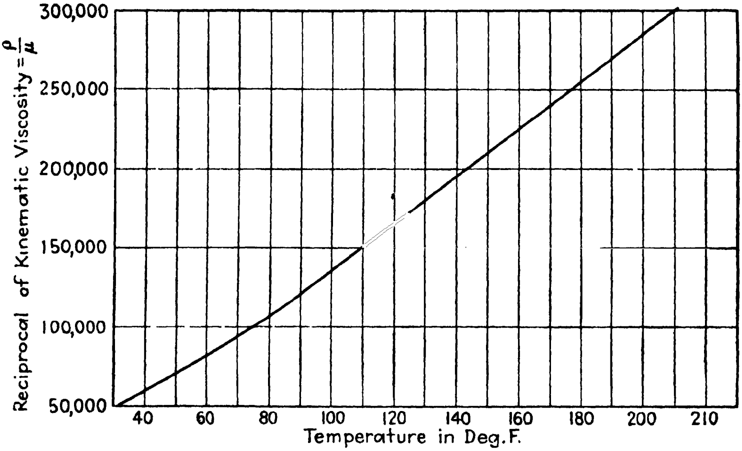

Fig. 13.- Relation between reciprocal of kinematic viscosity and temperature for water.

Figure 13 shows the relation between the reciprocal of kinematic viscosity and temperature for water, and Fig. 14 shows the relation between the friction factor *f* and the Reynolds number taken from a study by Kemler.[1] It appears that as the interior surface of pipes become rougher the value of *f* tends to become constant for all values of Reynolds number. This indicates that for very rough pipes the density and viscosity of the water have no effect on the loss of head due to friction.

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[1] Kemler, Emory, "A Study of the Data on the Flow of Fluids in Pipes, Univ. of Pittsburgh," *Trans. A.S.M.E.*, Vol. 55, 1933, HYD. 55-2.

Example: What will be the effect upon the friction loss in a 12-in. new steel pipe carrying water at 6 f p s. if the temperature of the water changes from 40 to 90°? (Interpolate values given in Table 7.)

Solution:

$$N_R \text{ at } 40^\circ = \frac{6 \times 1 \times 62.42}{0.001039} = 360{,}500, \qquad f = 0.0173$$

$$N_R \text{ at } 90^\circ = \frac{6 \times 1 \times 62.12}{0.000513} = 726{,}500, \qquad f = 0.0159$$

Answer: The friction loss would be decreased (0.0173 − 0.0159)/0.0173, or by about 8 per cent. The carrying capacity would be 8 per cent greater for the 90° water than it is for the 40° water.

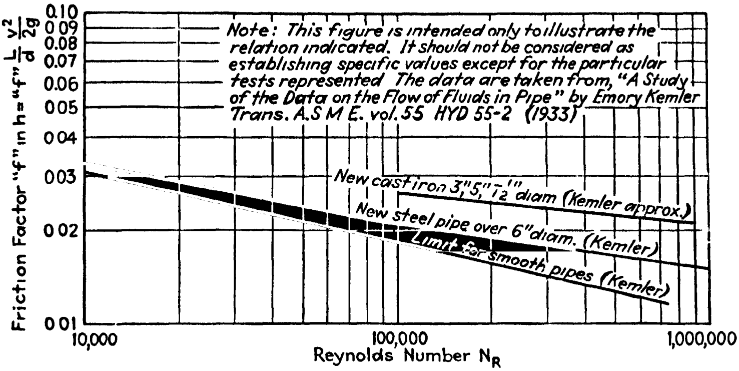

Fig. 14.—Relation between friction factor *f* and Reynolds number.

43. Minor Losses of Head. The principal head loss due to friction in a long pipe line results from the friction between the water and the sides of the pipe. This is the head loss indicated in the formulas in Table 3. However, other losses occur at changes in direction, pipe size, valves and fittings, and, in fact, at any point where turbulence is created. Some of the more common causes of minor losses of head are listed in Table 8.

TABLE 8.—MINOR LOSSES OF HEAD DUE TO FRICTION

Cause	Formula
Entrance to a pipe	$k_1 \frac{V^2}{2g}$
Sudden enlargement	$\left(\frac{a}{a_1} - 1\right)^2 \frac{V^2}{2g}$ or $\frac{(V - V_1)^2}{2g}$
Sudden contraction	$k_2 \frac{V_1^2}{2g}$
Around bends	$k_3 \frac{V^2}{2g}$

Nomenclature

k_1, k_2, and k_3 = coefficients
a = area of the larger pipe
a_1 = area of the smaller pipe
V = velocity in pipe, or in larger of two pipes
V_1 = velocity in smaller of two pipes

Values of k_1 depend on the character of the inlet. Values of k_2 are given in Table 9, and of k_3 in Table 10. The values shown in Table 10 are applicable primarily for small pipes up to about 2 in. in diameter. Their applicability to larger size pipes has not been demonstrated.

Loss of head through valves and fittings is commonly expressed in the form $h = k_nV^2/2g$. A few values of k_n are shown in Table 11.

TABLE 9.—VALUES OF k_2 FOR SUDDEN CONTRACTION

a/a_1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.00
k_2	0.362	0.388	0.308	0.267	0.221	0.164	0.105	0.053	0.005	0.000

TABLE 10.—VALUES OF k_2 FOR BENDS

D/radius of bend	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
k_3	0.131	0.138	0.158	0.206	0.294	0.440	0.661	0.977	1.40	1.98

TABLE 11.—VALUES OF k_n FOR FITTINGS[1]

Pipe fitting	k_n	Pipe fitting	k_n
Globe valve, wide open	10	Medium-radius elbow	0.75
Angle valve, wide open	5	Long-radius elbow	0.60
Close return bend	2.2	45-deg. elbow	0.42
Tee, through side outlet	1.8	Gate valve, wide open	0.19
Short-radius elbow	0.9	Submerged discharge	1.0

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[1] From LENDALL, H. N., "Hydraulics and Water Works Engineering," *Jour. A.W.W.A.*, December, 1939, p. 2075.

44. Hydraulic Grade Line. A hydraulic grade line is a line joining points whose vertical distances from the centroid of the cross section of a stream flowing in a closed channel are proportional to the pressure in the pipe at the point. In open-channel flow, the hydraulic grade line lies in the surface of the stream. The slope of the hydraulic grade line is known as the hydraulic gradient. The total energy line is the locus of points lying one velocity head above the hydraulic grade line.

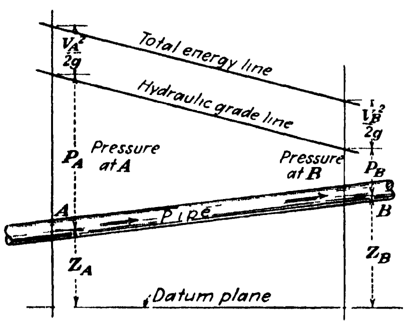

FIG. 15.—Bernoulli's theorem applied to closed-channel flow.

The hydraulic grade line and the total energy line, shown in Fig. 15, indicate the effect of friction losses and changes in velocity on the pressures and energy in a pipe line.

45. Equivalent Pipes and Compound Pipes. Two pipes, two systems of pipes, or a single pipe and a system of pipes are said to be equivalent when they have the same roughness coefficient and the losses of head for equal rates of flow in them are equal. Problems involving equivalent pipes are frequently encountered in waterworks practice.

TABLE 12.—EQUIVALENT PIPE CAPACITIES
(Based on Hazen and Williams Formula)

Diam. in.	4	6	8	10	12	16	24	36	48	Relation of cost to capacity[2]
4	1	......	......	......	......	......	......	......	.....	1.1
6	2.9	1	......	......	......	......	......	......	.....	0.52
8	6.19	2.14	1	......	......	......	......	......	.....	0.31
10	11.12	3.84	1.80	1						
12	18	6.21	2.90	1.61	1	......	......	......	.....	0.17
16	38.4	13.3	6.19	3.45	2.14	1	......	......	.....	0.12
24	111.2	38.4	18.0	10.0	6.19	2.90	1			
36	323	111.2	52.2	29.0	18.0	8 42	2.90	1		
48	689	238	111.2	61.9	38.4	18.0	6.19	2.14	1	
60	1,239	428	200	111.2	69.0	32.3	11.2	3.84	1.80	

The figures in the table show the relative rates of flow in pipes that will give the same loss of head with the same coefficient of roughness.

[1] From BOOTH, G. W., "Practical Studies of Distribution Systems," *Jour. A.W.W.A.*, September, 1944, pp. 957 and 961.

[2] From HOWSON, L. R., "The Distribution System," *Jour. A.W.W.A.*, December, 1943, p. 1517. Relation for 20-in. pipe is 0.09.

A compound pipe is made up of two or more pipes of different diameters connected end to end (in series) or of two or more pipes of the same or different diameters connected in parallel. The two conditions are illustrated in Fig. 16. Compound pipes exist in all water-distribution systems and in most long pipe lines. The solution of hydraulic problems in compound pipes depends upon a knowledge of equivalent pipes.

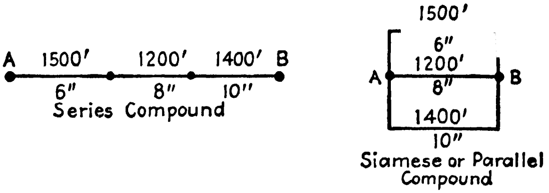

FIG. 16.—Compound pipe.

In the solution of problems involving rates of flow and head losses in compound pipes, the first step is to reduce the compound pipe to one equivalent pipe. In most computations it is customary to neglect minor losses of head. Any convenient value of C can be chosen since rates of flow vary directly with the value of the coefficient. In the following examples, C will be chosen as 100.

When the system is a series-compound, a procedure to determine the equivalent pipe is as follows:

1. Assume any convenient *rate of flow* through the pipe line that will give good intersections on the chart (Fig. 12). Compute the head losses in each section of pipe of the same diameter.

2. Find one pipe whose diameter and length are such that, with the assumed rate of flow, the head loss is equal to the sum of the head losses in the preceding step.

Example: Let it be desired to find a pipe equivalent to the series-compound pipe shown in Fig. 16.

Solution: It is found that an assumed rate of 400 g.p.m. gives good intersections on Fig. 12. Tabulating the computations:

Diameter of pipe, in.	Length in 1000's of feet	Head loss, ft. per 1,000 ft.	Total head, loss in, ft.
6	1.5	22.4	22.4 × 1.5 = 33.6
8	1.2	5.5	5.5 × 1.2 = 6.6
10	1.4	1.9	1.9 × 1.4 = 2.7
Total head loss			42.9

Since no length was specified for the equivalent pipe in the data, any convenient length may be assumed as, for example, 1,000 ft. Hence, the equivalent pipe has a head loss of 42.9 ft. per 1,000 ft. when carrying 400 g.p.m. The diameter of the equivalent pipe, read from Fig. 12, is, therefore, 5.2 in.

Likewise, the diameter of an equivalent pipe 2,000 ft. long, with head loss of 42.9/2 = 21.5 ft. per 1,000 ft., is read from the figure as 6 in.; *or* if the equivalent diameter is assumed as, say, 8 in., the head loss per 1,000 ft. is 5.1 ft. It would require 42.9/5.1 × 1,000 = 8,400 ft. of 8-in. pipe to be equivalent to the series-compound pipe shown.

A procedure[1] to be used when the system is parallel-compound is as follows:

1. Assume any convenient *head loss* between the junction points that will give good intersections on the chart (Fig. 12), and compute the rates of flow in each pipe.

2. Find one pipe whose diameter and length are such that with the assumed head loss the rate of flow is equal to the sum of the rates of flow in each pipe.

Example: Let it be desired to find a pipe equivalent to the parallel-compound system shown in Fig. 16. A head loss of 6 ft. from A to B will be assumed, for convenience. Then, tabulating the computations:

Diameter of pipe, in.	Length in 1,000's of feet	Head loss, ft. per 1,000 ft.	Rate of flow, g.p.m.
6	1.5	4	160
8	1.2	5	380
10	1.4	4.28	630
Total flow			1,170 g.p.m.

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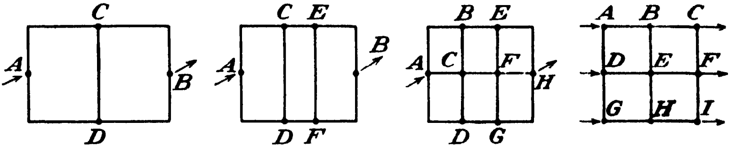
[1] Another method is offered by B. E. Seal, "Slide Rule Speeds Pipe Studies for Water Supply Systems," *Eng. News-Record*, June 12, 1947, p. 104.

If the length of the equivalent pipe is assumed to be 1,000 ft., then its diameter, with a flow of 1,170 g.p.m. and a head loss of 6 ft., is 12 in.; if the length of the equivalent pipe is assumed as 2,000 ft., its diameter is 14 in.; and if the equivalent diameter is 10 in., a length of 430 ft. must be assumed.

Other methods for the solution of problems in equivalent pipes have been suggested,[1] but their relative simplicities is dependent on the formulas used and the availability of tables for their solution.

46. Crossover Systems. A crossover occurs in a pipe system when two pipes are joined by one or more pipes that are connected to the other two between their points of joining. Single and multiple crossovers are illustrated in Fig. 17. Such crossovers are common in waterworks distribution systems. The head losses, rates of flow, and other factors cannot be computed by the equivalent pipe method. A method for the solution of such problems, devised by Hardy Cross, is explained in Sec. 388.



SINGLE DOUBLE MULTIPLE

FIG. 17.- Crossovers.

47. Fire Streams. The determination of the head loss, or rate of flow, in a single fire hose and nozzle attached to a hydrant presents no special difficulty when the coefficients of friction in the hose and of discharge in the nozzle are known. Ordinarily, fire hose is $2\frac{1}{2}$ in. in diameter and is rubber lined. The rate of flow through it can be expressed as

$$Q = kA\sqrt{2gh} \qquad (21)$$

where Q = rate of flow
k = coefficient of discharge of nozzle, usually lying between 0.95 and 0.99 for smooth conical nozzles
A = cross-sectional area of nozzle
g = acceleration due to gravity
h = pressure at base of nozzle

The loss of head through the fire hose may be computed by the Weisbach-Darcy formula in Table 3, using the value of f as shown in Table 13.

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[1] GRAVES Q. B., "A Simplified Solution for Equivalent Pipes," *Civil Eng.*, February, 1944, p. 70; and ARBUTHNOT, J. R., "Chart for Determination of Equivalent Pipes," *Water & Sewage Works*, August, 1946, p. 306.

TABLE 13.—FRICTION FACTORS IN 2½-IN. FIRE HOSE[1]

Description of hose	Values of f for velocities, f.p.s.				
	4	6	10	15	20
Unlined canvas	0.038	0.038	0.037	0.035	0.034
Rough rubber-lined cotton	0.032	0.031	0.031	0.030	0.029
Smooth rubber-lined cotton	0.024	0.023	0.022	0.019	0.018

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[1] From LENDALL, H. N., "Hydraulics and Water Works Engineering," *Jour. A.W.W.A.*, December, 1939, p. 207.

Two, three, or even four hose lines may be attached to the same hydrant. Where multiple hoses are connected to hydrants and the rate of discharge from one nozzle and the lengths of all hoses are known, the determination of the rates of flow and the pressures at the other nozzles must be solved by trial and error.

Example: Let it be required to determine the pressure at the hydrant, the pressure at the base of each nozzle, and the rate of flow from nozzle B in Fig. 18 if both nozzles are smooth, with $k = 0.97$, and the rate of flow from nozzle A is 356 g.p.m.

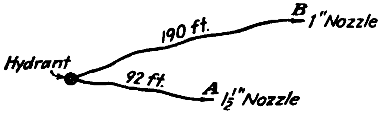

FIG. 18. Hose and nozzle problem.

Solution: 1. Determine pressure at the base of nozzle A.

$$h = \frac{Q^2}{2gk^2A^2} = \frac{\left(\frac{356}{7.5 \times 60}\right)^2}{64 \times 0.97^2 \times \left(\frac{2.25}{4 \times 144}\right)^2 \times 2.3} = 30 \text{ p.s.i.}$$

2. Determine the head loss in hose A.

$$h = f\frac{lV^2}{d2g} = 0.031\left(\frac{92}{\frac{2.5}{12} \times 64 \times 2.3}\right)\left(\frac{356}{60 \times 7.5 \times \frac{\pi}{4}\left(\frac{2.5}{12}\right)^2}\right)^2 = 50 \text{ p.s.i.}$$

3. Determine pressure at hydrant.

$$30 + 50 = 80 \text{ p.s.i.}$$

4. Assume Q from nozzle B = 200 g.p.m.
5. Determine pressure at nozzle B, using method in step 1. $h = 47.6$ p.s.i.
6. Determine loss of head in hose B, as in step 2.

$$h = 32.4 \text{ p.s.i.}$$

7. Determine pressure at hydrant, as in step 3.

$$47.6 + 32.4 = 80 \text{ p.s.i.}$$

This checks the pressure resulting from the known flow through nozzle *A*. Hence the assumed flow of 200 g.p.m. through nozzle *B* is correct. If it had not been, another assumption would be made and the procedure continued until a satisfactory result is found.

The rate of discharge from fire hydrants has been reported by Wilson[1] as

$$q = 27d^2p^{\frac{1}{2}} \tag{22}$$

where q = discharge, g.p.m.
d = internal diameter of hydrant nozzle, in.
p = pressure at hydrant, p.s.i.

Additional information on fire hydrants is given in Sec. 330.

The effective vertical height and horizontal distance of a jet for a good fire stream varies with the pressure at the base of the nozzle and size of the nozzle. Table 14 gives approximate values of height and horizontal distance of fire streams for different-size nozzles.[2]

TABLE 14.—EFFECTIVE HEIGHTS AND DISTANCES FOR FIRE STREAMS

Pressure at base of nozzle, lb per sq in.	½-in. nozzle		¾-in. nozzle		1-in. nozzle		1⅛-in. nozzle	
	Vertical height, ft.	Horizontal distance ft.	Vertical height, ft.	Horizontal distance, ft.	Vertical height, ft.	Horizontal distance, ft.	Vertical height, ft.	Horizontal distance, ft.
20	34	15	33	29	35	37	38	42
40	38	25	60	44	64	55	69	66
60	40	33	72	54	79	67	87	79
100	44	46	83	68	96	83	103	96

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[1] WILSON, P S, "Hydrant Discharge Measurements," *Water Works & Sewerage*, April, 1935, p. 115.

[2] FREEMAN, JOHN R., *Trans. A.S.C.E.*, Vol. 21, p. 303, 1889. FLEMING, V. R., *Univ. Ill. Expt. Sta., Bull.* 105, 1918.

CHAPTER III

DEMAND FOR WATER

48. Estimates of Demand. The making of an estimate of the probable rate of demand for water is among the first steps to be taken in the design of a waterworks system or the extension of an existing system. The determination of the demand is important because it fixes the capacities and sizes of all parts of the waterworks system. The rate of demand changes with the growth of the city, and it is obviously undesirable to construct a system, adequate for immediate needs, which might be outgrown in a short time. It is necessary, therefore, to make provision for future conditions.

The period into the future for which the estimate of demand should be made depends, to some extent, on the portion of the waterworks being designed and how it is to be financed. If the structure is long-lived and will be of benefit to future generations, it would be unfair to the present generation to put upon it the full financial burden. On the other hand, it is unwise and uneconomical to build for too short a period. In some states, 20 years is the legal limit of time.in which municipal bonds must be retired. Such a period is a fair guide to the judgment of the designer when no better information is available. By designing a structure to last during the period that payments are being made, the generation paying for it enjoys it, and no unfair financial burden is placed upon the present or upon the future.

In making an estimate of future demand, information on present consumption is frequently available. This may be in the form of pumpage records, meter readings, or other records. Such information is of great value. In using it, allowance should be made for the changed conditions that will result from the proposed improvement. Higher pressures, better quality, greater availability, and development of new uses for water, such as for air conditioning, will increase both the rate of demand and the total demand.

49. Population and Rate of Demand. Since water is supplied to the residents of a community for their use, it would seem to follow logically that the greater the number of users, the greater the use of water. However, because of many conditions affecting the rate of use of water, the rate of demand does not increase directly with the population. Among these conditions may be included the following:

1. In large cities nearly all persons are consumers; in small communities this is less likely to be true due to the private well or other source of supply.
2. In some cities the industrial demand may be relatively large; in others the industries may possess private sources of water supply.

When making estimates of demand based on the population of a small community, consideration must be given to such conditions. The daily per capita demand for water usually increases as the population of a city increases. This is due to greater industrial demands and to increased use for municipal purposes such as street cleaning, park irrigation, and fountain operation.

Studies reported by C. H. Capen, Jr.,[1] indicate that the following expressions may be used as an approximate indication of this trend:

$G = 53P^{0.11}$ for completely metered low-waste systems
$G = 54P^{0.125}$ for well-metered average systems
G = gallons per capita per day
P = population in thousands

50. Prediction of Population. Predictions of future events are logically based on knowledge of the past. Predictions of future population should be based on statistics of population and on knowledge of events affecting population. Information on the population of a community may be obtained from such sources as the United States decennial census, periodic school censuses, the chamber of commerce, the post office, public utilities, and health district records. In many engineering predictions of population growth it has been assumed that population trends of the past will be continued into the future, modified by environmental factors that can be foreseen at the time of the prediction. Five more-or-less "standard" methods have been used. They can be named as follows:

1. *Arithmetical progression,* in which a constant increment of growth is added periodically.
2. *Geometrical progression,* in which a constant percentage of increase is assumed for equal periods of time.
3. *Decreasing rate of increase,* which gives results between the arithmetical progression and the geometrical progression.
4. *Graphical extension* of the population-time curve into the future, by eye guided by judgment.
5. *Graphical comparison* with similar but larger cities.

The last method listed is the only one having any reasonable basis. In the use of this method the basic assumption is made that the city in question will grow as similar larger cities have grown in the past. Population-time curves are drawn to the same scale as the population-time curve of the city in question. The curves for the larger cities are then moved horizontally to the right until they intersect the curve of the city in question. Such a set of curves is shown in Fig. 19. These curves show how the larger cities grew after passing the population of the city in question. The curve of the city in question is extended into the future by eye based on the experience shown by the curves for the larger cities and assumptions concerning future events. It may be expected that this method will give the most reliable results of any of the statistical methods.

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[1] CAPEN, C. H., JR., "How Much Water Do We Consume?" *Jour. A.W.W.A.*, Vol. 29, p. 201, 1937; and "Water and Its per Capita Cost," *Water Works Eng.*, Feb. 17, 1937, p. 212.

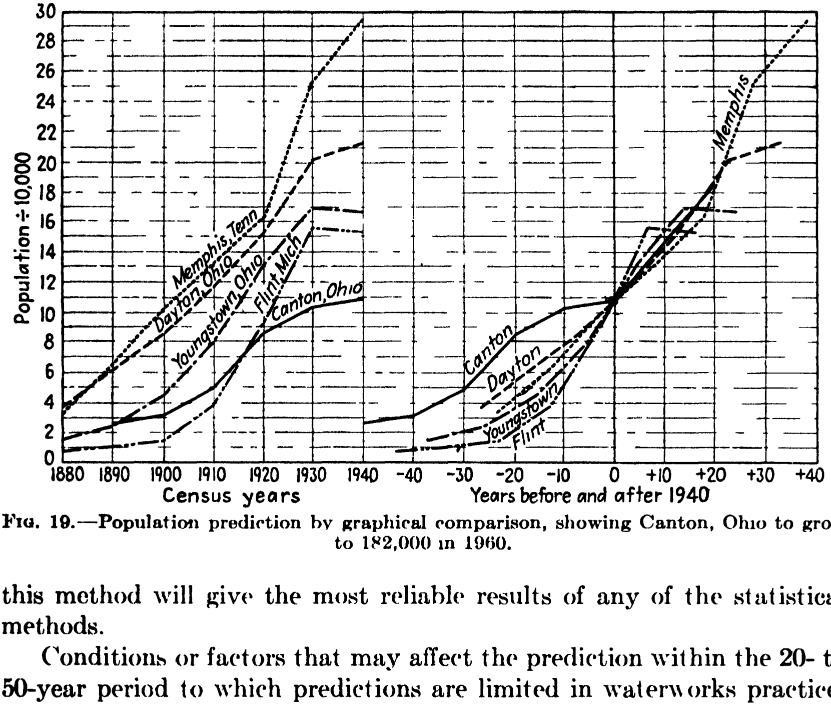

Fig. 19.—Population prediction by graphical comparison, showing Canton, Ohio to grow to 182,000 in 1960.

Conditions or factors that may affect the prediction within the 20- to 50-year period to which predictions are limited in waterworks practice, some of which cannot be expressed statistically, include the following:

1. Immigration rates, or population movements, trends of which may be observed by statistical studies.
2. Vital and morbidity conditions, revealed by a study of such statistics in the past.
3. Industrial and economic changes.
4. Educational, recreational, political, or other changes.
5. Transportation facilities.
6. War, disease, or other disaster such as fire.

Knowledge of the coming of some of these changes may be available. Engineering practice involves careful consideration and, where possible, a statistical analysis of all factors that may affect the prediction before a conclusion is reached. If the future brings changes not now predictable, it will be the responsibility of the waterworks administrators of the future to care for them.

51. Population Trends and Growths. Methods explained in the previous section for the prediction of population are based on the principle that population will increase indefinitely with time. Such a conclusion is obviously illogical. The population of the United States has increased fiftyfold in the past 150 years. The continuation of such a rate of growth is beyond the realm of probability.

Verhulst's[1] theory, paraphrased by Velz and Eich,[2] is:

> If population is expanding in a society of unlimited economic opportunity, the rate of increase is constant. If it is growing in an area of limited economic opportunity, the rate of increase must tend to get less and less as the population grows, so that the rate of increase is some function of the population itself, limited to a saturation value by the level of economic opportunity.

The United States is apparently entering an area of limited economic opportunity, and the population is approaching "saturation." If population statistics are converted to percentages of saturation (limiting number possible) and these figures are plotted on the logistic grid against time as abscissas, the curve will approach a straight line, the slope of which is known as the coefficient of growth. Examples of such plots are shown in Fig. 20. The logistic curve of Pearl[3] is a mathematical expression of Verhulst's theory, in the form

$$y = \frac{k}{1 + e^{a_0 + a_1 x}} \tag{1}$$

assuming three points uniformly spaced along the x axis, as $(O, y_0)(x_1, y_1)(2x_2, y_2)$,

$$k = \frac{2y_0 y_1 y_2 - y_1^2(y_0 + y_2)}{y_0 y_2 - y_1^2} \tag{2}$$

$$a_0 = \log \frac{k - y_0}{y_0} \tag{3}$$

$$a_1 = \frac{1}{x_1} \log \frac{y_0(k - y_1)}{y_1(k - y_0)} \tag{4}$$

The determination of the saturation value of the population is a matter of conjecture. It depends on the conditions of life at the time the saturation value is reached. If ratios of increments of increase of population for intervals of time are plotted against population at the ends of the periods, and the curve approaches the horizontal, saturation is approximated. As a result of studies on such a basis Velz and Eich[1] concluded that the great majority of our metropolitan centers have only 16½ per cent to go to reach saturation and that our smaller cities are within 86.2 per cent ±5.2 per cent saturation.

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[1] VERHULST, P. F., "Recherches mathematiques sur la loi d'accroissment de la population," *Mem. acad. roy. Bruxelles*, Vol. 18, , pp. 1–58, 1844.

[2] VELZ, C. S., and H. F. EICH, "How Old Are Our Cities," *Civil Eng.*, October, 1940, p. 619.

[3] PEARL, RAYMOND, "Studies in Human Biology," The Williams and Wilkins Company, Baltimore, 1924, p. 578.

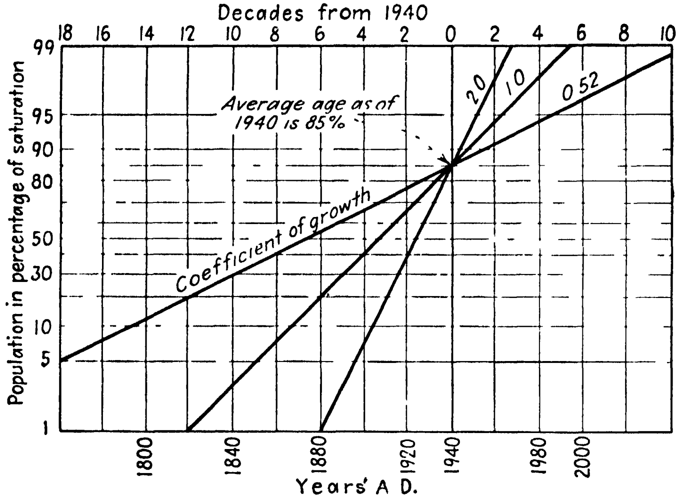

FIG. 20. - Logistic grid showing life span and time from 1940 to population stabilization of American cities over 25,000 (*From Civil Engineering, October*, 1940, *p.* 620.)

It is to be noted that growth characteristics are independent of size of community and that the coefficient of growth will indicate the probable time that the community will reach saturation. The coefficient of growth is a valuable tool in the prediction of the population growth of a community. Some statistics including coefficients and populations are shown in Table 15.

52. Rates of Water Usage. A man, under normal conditions of comfort and physical work, consumes as a beverage less than a pint of water a day. However, his demand for water for other purposes, on a desirable living standard, is relatively large. These other purposes that draw on the public water supply include, personally, the laundry, the kitchen, and the bath; and, communally, the industrial plant and the municipality.

Studies of statistics of the rates of usage of water in cities in the United States shows rates varying from a minimum of about 60 gal. per capita per day up to more than 300 gal. per capita per day. In rough approximation it is generally assumed that the average rate is in the neighborhood of 100 gal. per capita per day. No correlation seems to exist between rate of consumption and location, character of city, or the population except that it is usually assumed that the rate of use per capita per day is larger in large cities and in industrial areas. In European cities, rates are generally lower than in the United States, varying from 15 to 100 gal. per capita per day, under normal conditions. The lower rate of demand in European as compared with American cities is attributed to the greater percentage of dwellings abroad without modern plumbing, to the somewhat lower standard of living among the major portion of the populations of the countries of Europe, and also to the less wasteful attitude of the European householder.

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[1] VELZ and EICH, *op. cit.*

TABLE 15.—GROWTH CHARACTERISTICS OF SOME CITIES OF THE UNITED STATES[1]

City	Population in 100,000's	Saturation value in 100,000's	Per cent of saturation, 1940	Coefficient of growth
New York, N.Y.	109	160	91	1.31
Chicago, Ill	43.6	57	87	1.34
Los Angeles, Calif	23.2	35	85	2.4
Cleveland, Ohio	12.0	17	81	1.22
Baltimore, Md	9.5	13	77	0.71
Atlanta, Ga	3.7	5	83	1.29
Springfield, Ohio	0.687	0.80	90	1.10
Ogden, Utah	0.403	0.54	85	1.25
Danville, Ill	0.367	0.45	91	1.28
Bakersfield, Calif.	0.260	0.38	85	1.76

[1] VELZ, C. F., and H. F. EICH, "How Old Are Our Cities," *Civil Eng*, October, 1940, p. 619.

TABLE 16.—CONSUMPTION OF WATER FOR VARIOUS PURPOSES[1]

Use	Gallons per capita per day, average	Per cent of total
Domestic	37	34
Industrial and commercial	42	38
Public	11	10
Waste and miscellaneous	20	18
Total	110	100

[1] Approximate averages compiled from different sources, principally C. M. SAVILLE, *Jour. A.W.W.A.*, Vol. 7, p. 915, and FULLER and MCCLINTOCK, *Eng. Contr.*, Mar. 9, 1921, p. 222.

Allowance by the United States Navy for shore establishments[1] are, in gallons per capita per day: general average, 100; hospitals, 150 to 200; temporary camps without water closets, 75; hotels for naval personnel, 75; barracks for Waves, 125. Some Canadian statistics[2] are shown in Table 17. Berry states that in a tabulation covering 104 cities varying from less than 1,000 to more than 1,330,000, the average usage was 109 gal. per capita per day, with a low of 28 and a high of 427. The low occurred in East York with a population of 38,400 and the high at Merriton with a population of 3,000. It is to be noted that this information, and the statistics in Table 17, are contrary to the general trend of increase in rate of usage with increase in population.

TABLE 17.—CANADIAN RATES OF WATER DEMAND*

Population group	Number of cities	Average demand, gal. per capita per day
20,000 and over	31	114
5,000-20,000	40	103
Less than 5,000	31	110

* BERRY, A. E., "Some Canadian Water Consumption Statistics," *Water and Sewage*, November, 1940, p. 11.

In general, a rough approximation of 100 gal. per capita per day is commonly used in checking rates of water usage against population.

53. Use of Water for Specific Purposes. The purposes for which water may be used are classified in various ways. One classification is indicated in Table 16. The purposes named therein are self-explanatory except, possibly, in the case of use for the public. The city government uses water for flushing the streets; for fountains in parks, public buildings, schools; and for other distinctly municipal purposes. Water-waste surveys, the finding of leaks, and the prevention of waste are discussed in Secs. 410 to 413.

Statistics on the rate of use of water for specific purposes are listed in Table 18. In a study of 202 hotels St. Laurent[3] reports a use of 66 to 1,144 gal. per day per occupied room, with an average of 400.

54. Fire Demand. Protection against fire is one of the important functions of a waterworks. The total amount of water used in a year for extinguishing fires is usually a negligible part of the total consumption, but during a fire the rate of demand is so great as to be the deciding factor in determining the capacities of pumps, reservoirs, and distributing mains in all but the largest cities.

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[1] *Water Supply and Sanit., Circ. of Information, Bull.* Y. and D., March, 1945.

[2] BERRY, A. E., "Some Canadian Water Consumption Statistics," *Water and Sewage*, November, 1940, p. 11.

[3] ST. LAURENT, G. C., *Hotel Engineering*, 1940.

The fire demand is properly expressed in terms of the population, with a certain minimum limit, because the greater the population, the larger the number of buildings and the greater the possibility of fires. A residence burning in a village creates, however, as great a fire as when burning in a large city. The minimum limit of fire demand is, therefore, the amount that would be required to extinguish the largest probable fire that

TABLE 18.—RATES OF WATER USAGE FOR VARIOUS PURPOSES

Purpose	Amount
Domestic only, 10 cities in New York, New Jersey, and New England[1]	26–44.5 gal. per capita per day
Single-family houses, 1927:[1]	
West Haven, Conn.	36.8 gal. per capita per day
New Haven, Conn.	59.4 gal. per capita per day
Industrial:[2]	
Airplane engine, test	50,000–100,000 gal per unit
Buildings, office	27–45 gal. per capita per day
Hospital	125–350 gal. per bed per day
Hotels	306–525 gal. per guest room per day
Laundries	3–5.7 gal. per lb.
Restaurants	0.5–4 gal. per meal
Coke[2]	3,600 gal. per ton
Electric power	80 gal. per kwhr.
Milk, general dairy	340 gal. per 1,000 lb. raw milk
Strawboard	26,000 gal. per ton
Woolens	140,000 gal. per ton finished

[1] POND, M. A., "Urban Domestic Water Consumption," *Jour. A.W.W.A.*, December, 1939, p. 2003.

[2] JORDAN, H. E., "Industrial Requirements for Water," *Jour. A.W.W.A.*, January, 1946, p. 65.

TABLE 19.—RATES OF DEMAND FOR DOMESTIC CONSUMPTION
(Based on observations at Hartford, Conn.)

Type of dwelling	Persons per service	Gallons[1] per capita per day	Gallons per capita[2] per day		
			Maximum	Average	Minimum
Apartment houses	51.4	62.0	101	64	5
6-tenement houses	21.0	25.5			
3-tenement houses	12.5	28.4			
2-tenement houses	7.4	30.0			
3-family houses	4.5	31.2	37	15	4
2-family houses	5.4	45.7	66	34	11
1-family houses	4.3	81.4	75	54	15

[1] *Jour. A.W.W.A.*, Vol. 7, p. 926, 1920.

[2] *Jour. New Engl. Water Works Assoc.*, Vol. 27, p. 57, 1913.

could be started in a community. Knowledge of the community is necessary to fix this figure. A minimum of four fire streams available at all points within the district protected is desirable. Such streams should each throw at least 175 g.p.m. In rating a public water supply the National Board of Fire Underwriters fixes the required fire flow with respect to the population and the general structural condition of the district. The requirements vary between 1,000 g.p.m. for 1,000 persons to 12,000 g.p.m. for 200,000 persons. There must be enough water to care for a 10-hr. fire at not less than 20 p.s.i. pressure at the hydrant with allowance for possible broken pipes.[1] Even these rates are not so high as those demanded by a conflagration like that at Salem, Mass., in 1914. Here a city of about 50,000 population needed about 17,000 g.p.m. for about 14 hr.

Various authorities have made recommendations concerning the proper allowance for peak fire demand when designing a waterworks system. These recommendations are shown in Table 20.

TABLE 20.—EMPIRICAL FORMULAS FOR RATES OF FIRE DEMAND

Name of originator or authority	Formula: Q = demand in gallons per minute P = population in thousands	Gallons per minute for 100,000 population
Kuichling (on basis of fire streams of 250 g.p.m)	$Q = 700\sqrt{P}$	7,000
Freeman, John R.	$Q = 250\left(\frac{P}{5} + 10\right)$	7,500
National Board of Fire Underwriters	$Q = 1{,}020\sqrt{P}\,(1 - 0.01\sqrt{P})$	9,180

The capacities of pumps and distributing mains are affected by the rate at which water must be supplied to the fire. The capacities of reservoirs are fixed by the duration of the fire. In small cities a duration of 6 hr. is assumed, as suggested by Freeman. The National Board of Fire Underwriters recommends an allowance of 5 hr. for towns of less than 2,500 population and 10 hr. for larger cities.

55. Fluctuations in Rates of Demand. The maximum rate of usage of water varies greatly with the time over which the rate is computed. The shorter the period of observation, the greater the deviation from the long-period average. Deviations of maximum rates from the annual average are indicated in general terms in Table 21.

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[1] See also GOLDSMITH, C., "The Relation of Fire Protection Requirements to the Distribution System of Small Towns," *Jour. A.W.W.A.*, Vol. 12, p. 168, 1924.

TABLE 21.—AVERAGE RATES OF DEMAND FOR VARIOUS PERIODS OF TIME[1]

Period	Maximum rate as per cent of average annual rate	Per cent of average day in 10 American cities[1]
Annual, average	100	
Season, maximum	130	
Month, maximum	140	
Day, maximum	150-180	172
Hour, maximum	250-350	280
Hour, minimum	.....	70

[1] From KINCAID, R. G., "Analyzing Distribution Systems," *Water Works Eng.*, Mar. 22, 1944, p. 288.

Seasonal rates in temperate climates are usually above average in the summer and winter. In summer, water consumption is high due to lawn watering, bathing, and cooling. In winter it may be high due to waste of water to prevent freezing. Monthly rates in some Massachussetts cities are shown in Table 22. Daily and hourly rates of demand are shown in Figs. 21 and 22. The highest published rate of water use, exclusive of fire demand, occurred in Chicago in the summer of 1940[1] when the rate for 1 hr. was 492 gal. per capita per day, and for the maximum day 414 gal. per capita; although the total pumpage of 1,414 million gal. on Aug. 6, 1947, is the highest daily pumpage recorded for the city.

TABLE 22.—CONSUMPTION OF WATER DURING DIFFERENT MONTHS[1]
(Averages for 10-yr. period in Massachusetts cities)

Month	Annual average percentage	Month	Annual average percentage
January	90	July	124
February	92	August	118
March	89	September	108
April	87	October	96
May	100	November	90
June	113	December	92

[1] *Jour. New Engl. Water Works Assoc.*, Vol. 27, p. 85, 1913.

Hydrographs such as those shown in Figs. 21 and 22 show some of the habits of the water users. In general, rate of demand is the greatest during morning hours and on Monday. The lowest rate of usage falls on

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[1] "Chicago Water Demand Reaches All-time Peak," *Water Works Eng.*, Aug. 28, 1940, p. 1098.

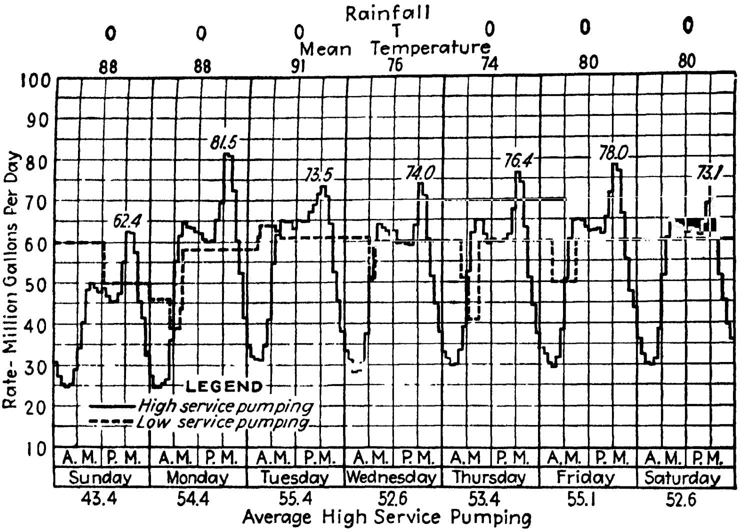

FIG. 21.—Demand curve for week of July 12 to 18, 1936, at Toledo, Ohio. (*Courtesy of Burns and McDonnell Engineering Company.*)

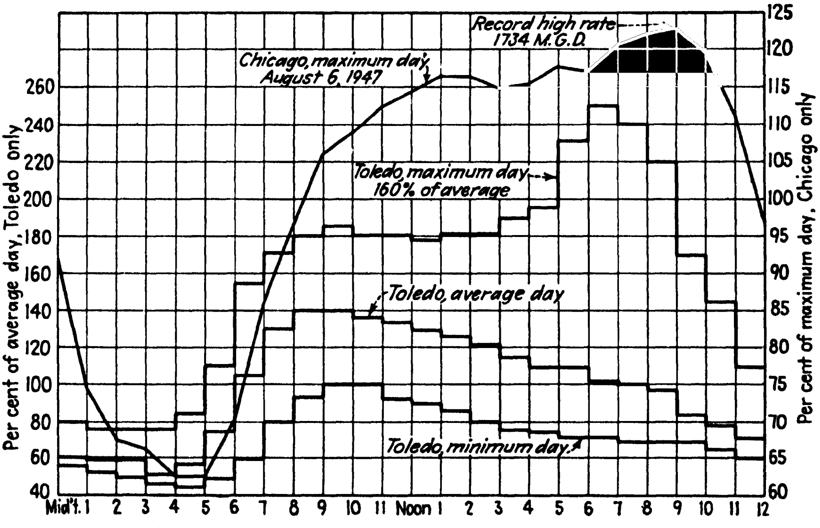

FIG. 22.—Hydrographs of hourly pumpage for typical maximum, minimum, and average days at Toledo, Ohio, 1937 (*courtesy of Burns and McDonnell Engineering Co.*), and for maximum day at Chicago, Ill., Aug. 6, 1947 (*courtesy of L. D. Gayton*).

Sunday, the extreme minimum falling in the small hours of Monday morning. A hydrograph of water demand for Toledo, Ohio, representing a typical industrial city, is shown in Fig. 22. Typical fluctuations of rate of consumption in a small city are shown in Fig. 23. A typical weekly chart showing hourly variations at Springfield, Mass., is shown in Fig. 24.[1] A careful study of this figure will tell much concerning hourly and daily variations in demand. For example, the area under the curve when read in correct units represents the total flow between the hours observed.

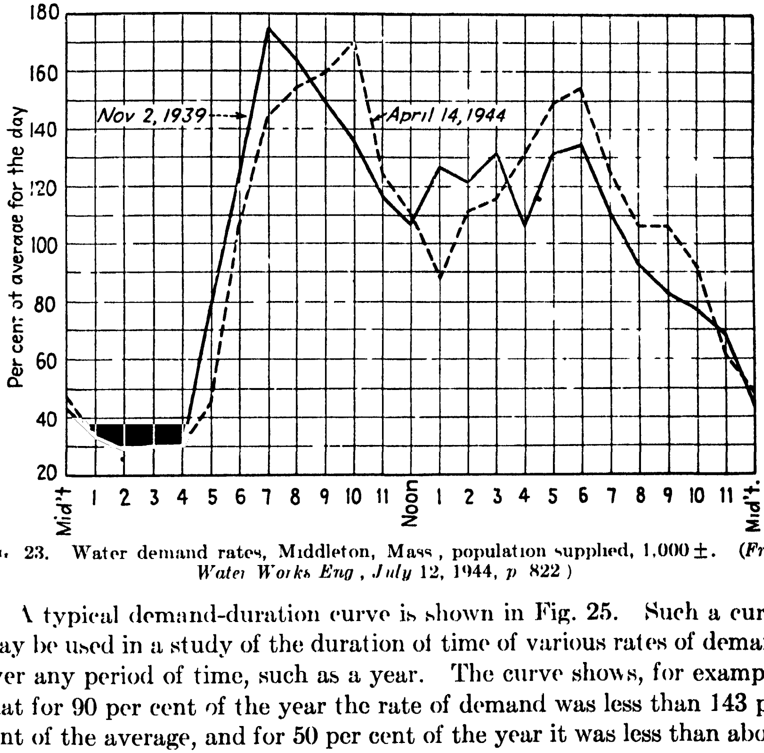

Fig. 23. Water demand rates, Middleton, Mass., population supplied, 1,000 ±. (*From Water Works Eng.*, *July* 12, 1944, *p.* 822.)

A typical demand-duration curve is shown in Fig. 25. Such a curve may be used in a study of the duration of time of various rates of demand over any period of time, such as a year. The curve shows, for example, that for 90 per cent of the year the rate of demand was less than 143 per cent of the average, and for 50 per cent of the year it was less than about 96 per cent of the average rate of demand.

56. Conditions Affecting Rate of Demand. Rates of water consumption are affected by the size of the community, its location, the standards of living of the population, the pressure maintained on the distribution system, the quality and the cost of the water, the existence of sewers, the extent of the use of meters, and other factors. Each of the factors is of considerable importance in determining the rate of water consumption and, in an estimate of demand, should be considered independently.

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[1] From Richardson, C. G., "Moving Finger Shows Flow Rates at Springfield, Mass.," *Water Works Eng.*, July 28, 1943, p. 831.

57. Population and Living Standards. Rates of water use as affected by population are stated on page 51. Rates of use in various classes of districts are shown in Table 23.

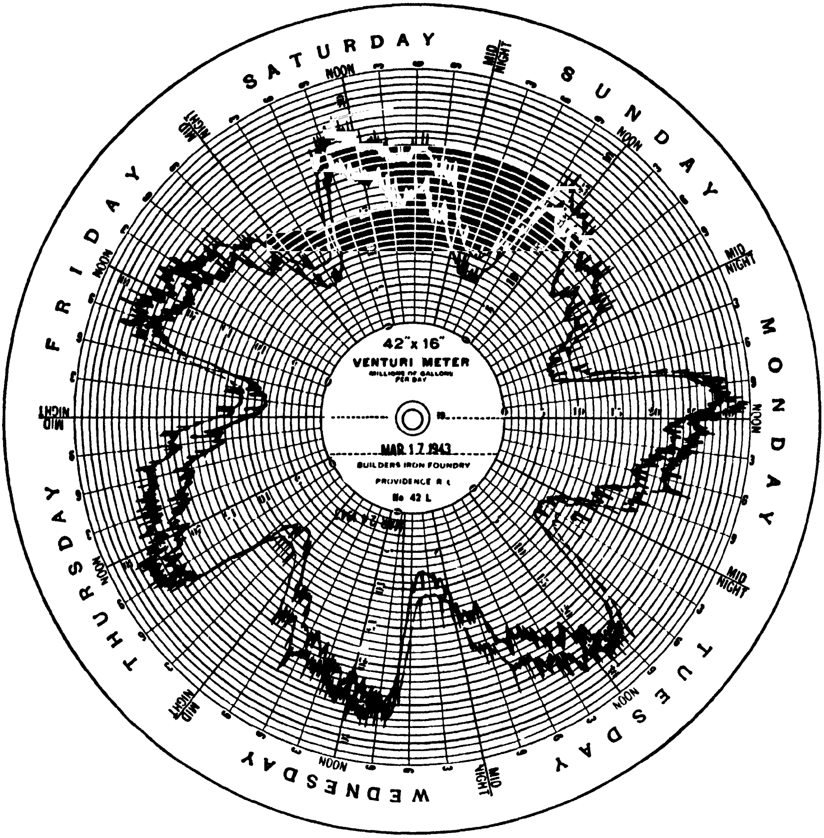

FIG. 24.—Hourly variations in rate of pumping for week of Mar. 17, 1943, at Springfield, Mass. Inside curve represents week of Mar. 24, 1940, and outside curve represents week of Mar. 17, 1940. (*From C. G. Richardson, Water Works Eng., July 28, 1943, p. 831.*)

58. Location and Climate. The per capita rate of water use in cities in the United States is higher than elsewhere in the world. Although wastefulness is probably a factor, the facilities for the use of water in American homes are more abundant, the pressure and quality of water is generally good, and standards of living are on a high level. When comparison is made between European and United States communities with similar characteristics the difference is not marked. For example, Munich, with an excellent water supply, reported 54 gal. per capita per day as compared with San Francisco reporting 65 gal. per capita per day.

Rates of use in cities in the northern part of the United States are greater than in cities of similar size in the South. This fact offers a clue to the difference between the rates of water consumption in European and American cities. The temperature variations in northern United States are greater than in the South and in Europe. During protracted hot periods, water is used for gardens, lawns, and cooling purposes, and during extremely cold weather it is allowed to run to avoid freezing. In cities where gardens and lawns are watered, particularly in arid climates, as in parts of Colorado and the Southwest, the Indian Punjab, and parts of South Africa, the rates of water consumption will be large when compared with similar cities in districts with more equable rainfall. In humid, tropical, insular climates, as at Havana, Cuba, the average annual rate of consumption of 80 gal per capita per day is lower than that in most cities of similar size in higher latitudes in the United States.

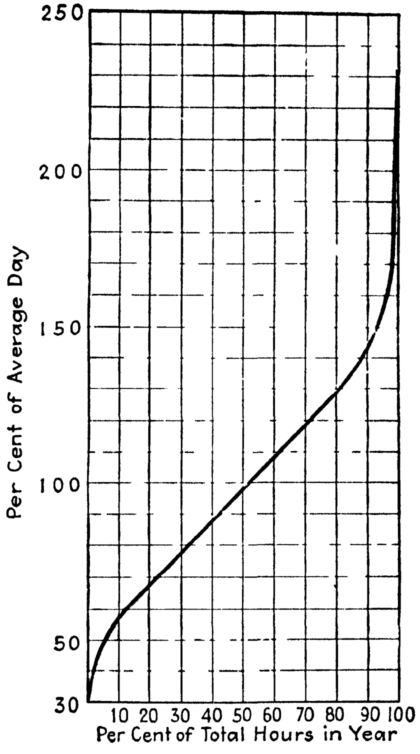

FIG. 25 —Demand-duration curve.

59. Character of District. Cities and districts of cities can be classified as industrial, commercial, and residential. Residential districts can be classified as high class, medium class, and poor and also according to the predominating racial or antecedent characteristics of the population. The characteristics of the district will be reflected in the rate of water consumption therein. The rates of consumption in different classes of districts are shown in Table 23. Those for domestic purposes will represent the consumption in residential districts. These districts can be classified and the rates of consumption in the different classes of districts studied. It is found that the standard of living materially affects the rate of water consumption. The standard of living in a district is indicated by the appearance of the homes and their surroundings and by the density of the population. The better the homes and the less dense the population, the greater is the rate of water consumption per capita.

TABLE 23.—RATES OF WATER DEMAND FOR DOMESTIC CONSUMPTION AT NEW HAVEN CONN., IN 1937, SHOWING THE EFFECT OF THE CHARACTER OF THE DISTRICT[1]

Economic status	Single-family houses			Multiple-family houses		
				Gallons per capita per day		
	No. of houses	Average persons per family	Use, gal. per capita per day	Two family	Three family	Four family
Excellent	290	3.13	125	44.2	41.6	52.0
Excellent	162	3.20	85	39.7	42.2	43.1
Good	66	3.07	55.7	42.3	51.2	44.6
Good	335	3.77	38.0	27.5	37.6	27.0
Fair	176	2.92	39.0	37.0	38.6	45.9
Fair	470	3.82	33.9	31.0	31.9	36.7
Poor	20	4.34	54.0	34.2	35.7	29.3

[1] From POND, M. A., "Urban Domestic Water Consumption," *Jour. A.W.W.A.*, December, 1939, p. 2003.

60. Pressure and Quality. The rate of water consumption is increased when the pressure is increased. This is due to the greater loss through leaks and the greater amount run to waste through open faucets. Increases in rates of water consumption with increases in pressure have been known to reach 30 per cent for a change from 25 to 45 p.s.i. This fact should lead the designer to provide the lowest pressure that will give satisfactory service. Excess pressure means waste.

The improvement of the quality of the public water supply will result in an increase in water use due to the availability of the water for more uses and a feeling of safety on the part of the public using it.

61. Intermittent Pumping. If water is supplied for less than 24 hr. daily in order to diminish the rate of consumption, the results are likely to be disappointing, and other evils are likely to develop. Waste is so greatly increased by an intermittent supply that the consumption of water may be greater than when it is supplied continuously.

When water is supplied for less than 24 hr. per day, and the supply is not metered, it becomes the custom of the consumers to place receptacles under faucets and hydrants and to leave these services open during the time of nonsupply so that the receptacles may be filled when the supply is turned on. The receptacles remain unattended and, when filled, overflow to waste. Just before the resumption of supply is expected, the water remaining in the receptacles will probably be thrown away in order to assure a fresh and clean supply in the receptacle.

62. Air Conditioning. Water requirements for air-conditioning purposes can be calculated for individual installations, but it is difficult to predict what the over-all future demand will be. Gayton[1] predicted in 1937 that within the next 20 years the demand for air conditioning in Chicago might reach a peak load of 300,000,000 gal. per day.

TABLE 24.—AVERAGE CONSUMPTION OF WATER BEFORE AND AFTER THE INTRODUCTION OF SEWERS[1]
(Gallons per capita per day)

City	Years previous to the introduction of sewers							Years after the introduction of sewers									
	7	6	5	4	3	2	1	1	2	3	4	5	6	7	8	9	10
Marlborough, Mass.	13	17	20	21	24	24	25	30	30	35	34	37	37	38	38	36	37
Newton, Mass.	28	31	33	33	31	36	40	50	52	60	65	63	60	57	63	62	63
Waltham, Mass.	37	36	39	33	31	32	33	47	53	61	59	71	70	76	88	90	88
Madison, Wis.	Unsewered residences 14 gal. per capita daily							Sewered residences 68 gal. per capita daily									

[1] Report of Committee on Water Consumption Statistics and Records, *Jour. New Engl. Water Assoc.*, Vol. 27, p. 70, 1913.

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[1] GAYTON, L. D., "Air Conditioning as a Problem of Water Distribution and Disposal," *Jour. A.W.W.A.*, Vol. 29, p. 808, 1937; and "Air Conditioning in the City of Chicago," *Jour. A.W.W.A.*, Vol. 30, p. 892, 1938; H. E. BABBITT, "Developments in Water Works Practice," *Jour. A.W.W.A.*, Vol. 30, p. 454, 1938.

It is estimated by Lewis[1] that between 1945 and 1950 the maximum rate may vary from 23 to 35 gal. per capita per day and the average rate from 3 to 5 gal. per capita per day. The variation depends upon whether cooling towers or evaporative condensers are used, and the figures are based upon the rate of sales of equipment.

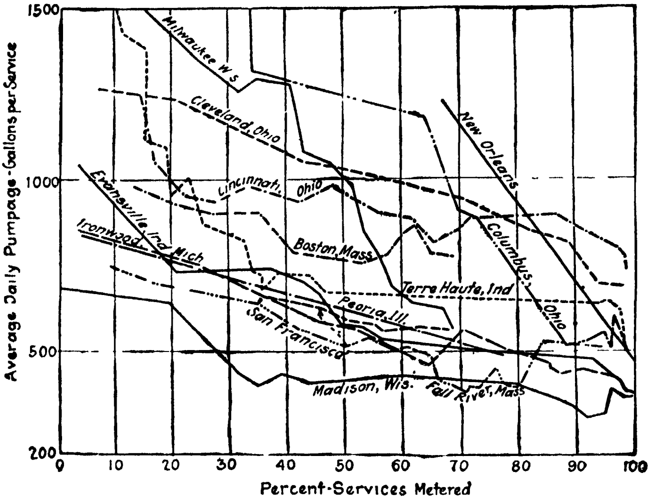

FIG. 26.—Diagram showing effect of meters on pumpage of water per service. (*Courtesy Alvord, Burdick and Howson, Engineers, Chicago,* 1925.)

The demand for air-conditioning water is seasonal and is affected by the summer temperatures that prevail. In many installations the cooling water is derived from private wells; hence such systems do not directly create a demand on a municipal supply.

TABLE 25.—RELATION BETWEEN DEMAND AND COST OF WATER[1]

Percentage increase in cost	Percentage decrease in rate of water consumption
20	13
40	22
60	29
80	35
100	40

[1] LEONARD METCALF, "Effect of Water Rates and Growth in Population upon per Capita Consumption," *Jour. A.W.W.A.*, Vol. 15, p. 1, 1926.

63. Sewers. The effect of the installation of sewers in a district is to increase the rate of water consumption because of the increase in plumbing facilities, as indicated by the figures in Table 24.

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[1] LEWIS, LOGAN L., *Water Works & Sewerage*, May, 1937, p. 188.

64. Cost of Water. The rate of water consumption diminishes with increased cost of water. This is indicated by the figures in Table 25.

65. Effect of Meters on Demand. It is common belief that the metering of a water supply diminishes the demand for water. A diagram showing this effect is shown in Fig. 26. In unmetered communities, where water is paid for at a flat rate, there is no incentive to conserve water and more is used than is required.

66. Policy of Metering. Arguments for and against metering sometimes develop more heat than light. A summary of some points on each side of the question is presented in Table 26.

TABLE 26.—SOME POINTS FOR AND AGAINST METERING

For	*Against*
1. It is only just that the consumer should pay in proportion to the amount he uses	1. Limited use of water may result in unhygienic conditions and disease.
2. Waste is diminished, resulting in financial saving to all	2. Meters cost money to buy, to install, to maintain, to read, partly defeating their own purpose
3 The poor actually pay more through taxes where meters are not used	3. Pressure losses through meters are appreciable and add to pumping cost
4. Loads on purification plants and pumps, etc., are minimized	4. The poor suffer more than the rich through water charges
5. Waste surveys are easier to conduct; leaks are more easily found	5. Consumer resents stand-by charge made when meter shows no water used
6. Most privately owned waterworks are 100 per cent metered[1]	6. The use on gardens is diminished, affecting economy and appearence of community
7 The careful consumer benefits, the careless consumer is penalized	7. Meter money might better be spent on waterworks improvement
	8. Waste can be more economically checked by inspection than by meter

[1] It is illegal in Nevada for a private company to sell water by meter. (See *Water Works Eng.*, Feb. 26, 1941, p. 211.)

The argument that meters require pressure for their operation is valid. Specifications of the American Water Works Association for cold-water meters permit a head loss of 20 p.s.i. at rated capacity. Tests[1] show the head loss to vary approximately as the square of the rate of flow through the meter. If the cost of pumping water and the amount of water used through a meter are known, the cost of operating the meter can be computed approximately. The argument that meters are costly is likewise valid. In 1940, a 5/8 in. meter cost $12 to $18 installed.

Waterworks engineers generally favor the metering of water supplies where total cost of the waterworks is an important factor and the quantity of water available at the source is limited.

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[1] "Water Meters Should Be Selected with More Care," *Eng. News-Record*, Dec. 12, 1918, p. 1074; and GARRATT, J. E., "Testing Water Meters in Series," *Jour. A.W.W.A.*, Vol. 8, pp. 127 and 130, 1921.

CHAPTER IV

GROUND WATER

67. Occurrence of Ground Water. A portion of the rain that falls on the ground percolates through it and fills the interstices of the substrata until the water appears again at the surface in the form of springs or other ground-water overflow. The factors affecting the amount of rainfall that may percolate into the ground include the amount and intensity of the rainfall and its distribution, the topography and the permeability of the ground surface, and the water capacity of the ground.

The surface of the ground water that is under atmospheric pressure is known as the "ground-water table." Water may be held also in a pervious stratum under an impervious stratum, at a pressure greater than atmospheric. A water-bearing stratum is sometimes spoken of as an *aquifer*. The ground-water table does not remain constantly at the same elevation. It fluctuates with the rainfall, temperature, barometric changes, tides, and the rate of pumping from wells. Practically all ground water is in motion. Hence its surface is not level, its slope depending on the imperviousness of the material and the rate of flow of water through it. The shape of the ground-water table follows, roughly, the topography of the ground-water surface, as shown in Fig. 27. A knowledge of the conditions under which ground water occurs and of the physical laws of the flow of ground water is essential to the successful development of ground-water supplies.[1] In studying the subject of ground-water occurrence and use, the student should consult the appropriate *Water Supply Papers* of the U.S. Geological Survey.

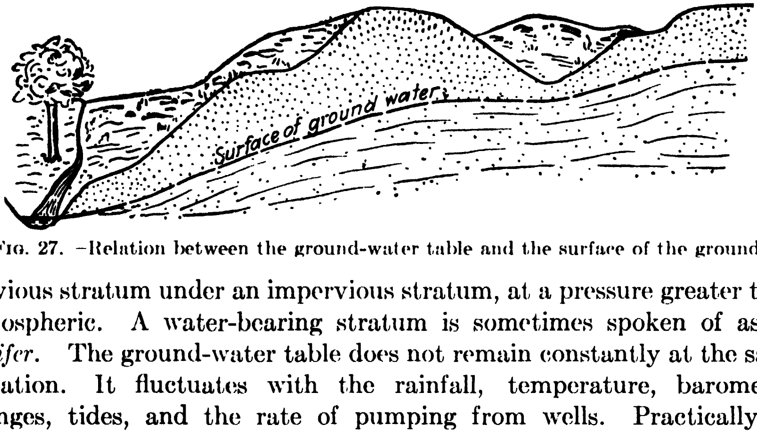

FIG. 27. —Relation between the ground-water table and the surface of the ground.

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[1] See also *U.S. Geol. Survey*, *19th Ann. Rept. Part* II, p. 381, 1897–1898; *U.S. Geol. Survey*, *Water Supply Paper* 120, 1903; and *U.S. Geol. Survey*, *Water Supply Paper* 67, 1902.

68. Water-bearing Formations. Ground water occurs in three principal forms: (1) in buried river valleys or in the beds of ancient lakes, (2) in extensive sedimentary deposits such as occur under the prairies and Western plains in which the water may be held in porous rock or buried under pervious strata, and (3) in porous strata in which the resistance to flow together with the rate of percolation from the surface are sufficient to maintain a reservoir of ground water well above the elevation of the natural surface outlet. The occurrence of ground water in free-flowing underground streams, as in caverns in limestone districts, is rare but in some localities furnishes large quantities of water.

Sand and gravel are materials in which ground water is most commonly found available for public water supplies. The porosity of natural sands or gravels may be as high as 30 to 35 per cent.[1] Igneous rocks have a pore space of about 1 per cent; shale, 4 per cent; limestone, about 5 per cent; American oölites, between 3 and 12 per cent, averaging about 7 per cent; sandstones, 13 to 30 per cent; and some clays as high as 50 per cent. In this connection it is necessary to distinguish between porosity and perviousness, for it is upon the latter that the availability of water from underground strata depends. *Porosity* is the volume of the pores of a substance. It is usually expressed as a percentage of its total volume. It represents the volume of water that the dry material will soak up. *Perviousness* is the capacity of a substance to allow water to pass through it, which is equivalent to the water available for a supply. Although some clays may be 50 per cent porous, they may be practically impervious, whereas an igneous rock only $\frac{1}{2}$ to 1 per cent porous may yield all of its water. Nearly all the porosity in coarse materials is available for water storage. Large cracks or channels in an underground stratum, through which water may flow, should not be confused with the pores and interstices of the material which determine its perviousness.

Permeability and *transmissibility* express the relative ease with which water will flow through a porous medium.[2] The coefficient of permeability may be expressed as the gallons of water which, in 1 day, will flow through 1 sq. ft. cross-sectional area of the sample of material, when the hydraulic gradient is unity and the temperature is 60°F. The coefficient of transmissibility is the gallons of water which, in 1 day, will flow through a vertical strip the height of the aquifer and 1 ft. wide, measured at right angles to the direction of flow when the hydraulic gradient is unity. The coefficients of permeability and of transmissibility are not usually equal since the former is determined on a sample in the laboratory and the latter on the undisturbed aquifer in the field. Predictions of underground flow conditions based on transmissibility may, therefore, be expected to be more dependable.

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[1] SMITH, B., "Geological Aspects of Underground Water Supply," *Water and Water Eng.*, April, 1936, p. 223.

[2] See also SUTER, MAX, "Permeability and Transmissibility, What Are They?' *Illinois Well Driller*, Vol. 15, pp. 4, 13, 1946.

Water tables are fairly easy to define and locate in the comparatively unaltered stratified rocks, but they are by no means so easy to determine in those impervious or relatively impervious rocks that are exemplified by the more ancient strata of this country. These early strata include metamorphic rocks such as schists and those possessing a slaty cleavage. If the rocks of this type are highly jointed, however, the water filling the fissures may rise to about the same general level over a wide area and so give an approximation to a water table.

The size of the sand grain has no relation to the amount of available water stored, but coarse-grained sand delivers its water to a well with less trouble than fine-grained material. Sandstones are used as sources of large water supplies, particularly in artesian areas, where the deep-lying sandstones contain water under pressure. The most common material in which ground water is encountered is *till*, a mixture of clay, sand, gravel, and boulders. It may hold large quantities of water and serve as a very satisfactory source of supply. Clay, shale, granite, and gneiss are almost impervious to water, and wells sunk in such material cannot be expected to yield large quantities. Limestone is not highly porous, but it contains caverns and channels which may hold large quantities of water. Water in rocks may carry pollution to some distance through cracks and interstices too large to serve as filters.

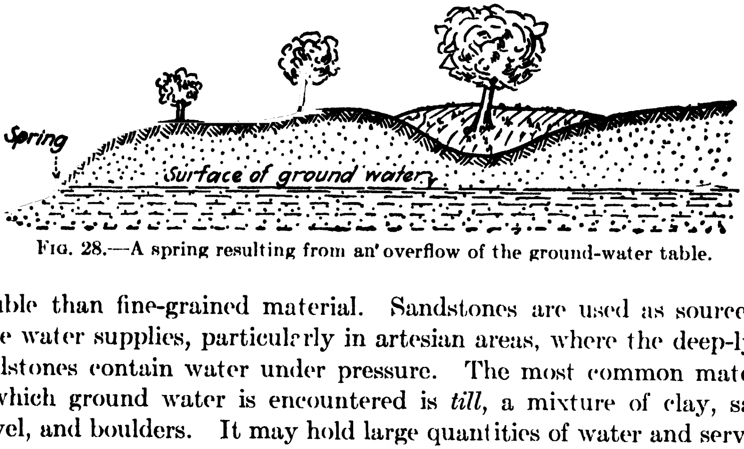

FIG. 28.—A spring resulting from an overflow of the ground-water table.

69. Springs. Ground water reappears at the surface in three different types of springs. The first results from an overflow of the ground-water table, as shown in Fig. 28. The second results from the encountering of an outcropping impervious stratum by percolating water, as shown in Fig. 29. The third results from the escape, at the surface, of percolating water which has been held under pressure beneath an impervious stratum. This is known as an artesian spring, and the type is illustrated in Fig. 30.

The greatest amount of water and the greatest certainty of continuous flow from springs of the first type can be secured by constructing a cutoff trench along a contour at or near the point of appearance of the spring. The deeper the trench, the greater the certainty of continuous flow, as the saturated ground above the trench bottom will act as an impounding reservoir to compensate for the fluctuations of the groundwater table. A trench, or infiltration gallery, to serve such a purpose is shown in Fig. 31.

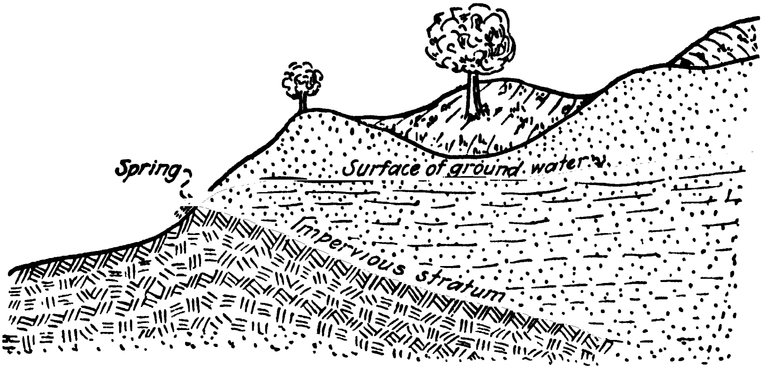

FIG. 29.—A spring resulting from an outcropping of an impervious material.

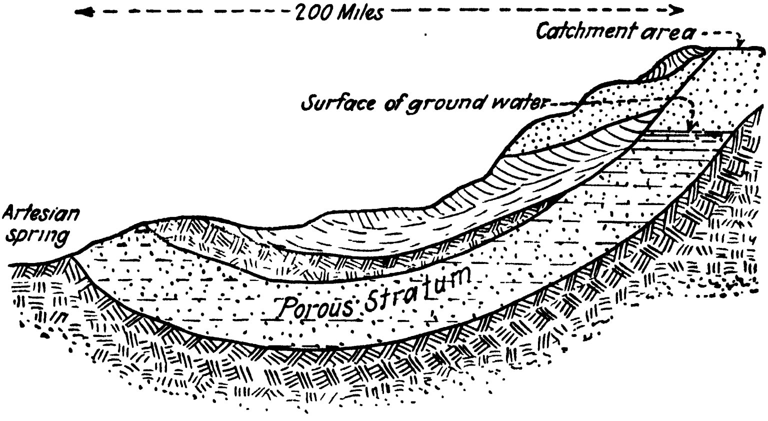

FIG. 30.—Conditions resulting in an artesian spring.

Springs of the second type, shown in Fig. 29, occur more frequently than other types of springs. Because of the relatively small amount of underground storage available, the flow from them is uncertain and likely to cease after prolonged drought. Such springs are best developed by the construction of a cutoff trench. There is no advantage in constructing the bottom of the trench below the surface of the impervious stratum, as no more water will be secured in this manner.

Artesian springs usually have a large tributary watershed at some distance from the spring, and the water may appear under pressure. The amount of water available in such springs is usually large, and the flow may be slightly increased by removing obstructions from the mouth of the spring. Extensive artesian areas exist on all continents. Large flowing fresh-water springs have been discovered miles out in the Atlantic. The artesian wells of the Dakotas are noted for their high pressures which have reached 250 p.s.i. at the surface in some wells.

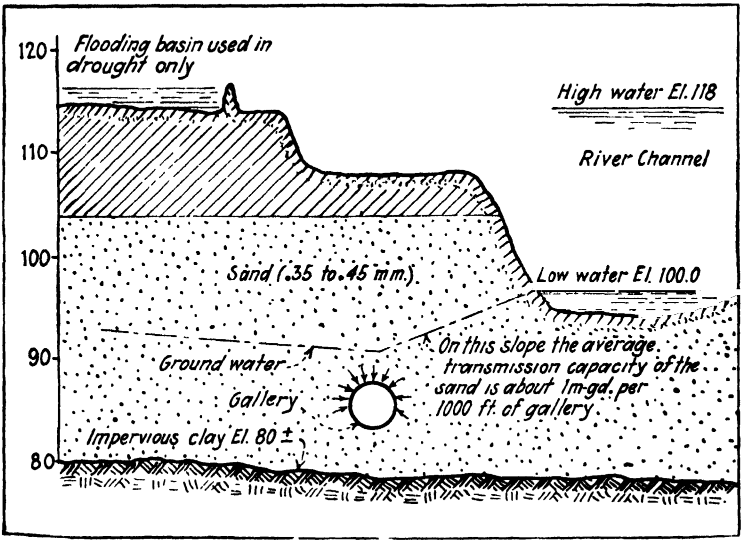
120
110
100
90
80
Flooding basin used in drought only
High water El. 118
River Channel
Sand (.35 to .45 mm.)
Low water El. 100.0
On this slope the average transmission capacity of the sand is about 1m-gd. per 1000 ft. of gallery
Ground water
Gallery
Impervious clay El. 80 ±

FIG. 31.—Typical section of filter gallery, Des Moines, Iowa. (*From Public Works, Vol.* 55, *p.* 332, 1924.)

Springs of great magnitude issue from various kinds of rocks. O. E. Meinzer has classified springs according to the rate of discharge from them, as shown in Table 27. Of 65 springs of the first magnitude in the United States, 37 issue from volcanic rock and from gravel, 24 from limestone, 3 from sandstone, and 1 from a lake bed overlying volcanic rock.[1] Possibly the largest spring in the world, based on the maximum rate of flow, is the famous Fontaine de Vaucluse in France. It has a maximum discharge of 5,300 c.f.s. and an average flow of 800 c.f.s.

The reverse action to the appearance of a spring is the disappearance into the ground of a surface stream. This phenomenon appears most commonly in limestone regions and in sandy areas. A large part of the

[1] *U.S. Geol. Survey, Water Supply Paper* 557, 1927.

Colorado River disappears into the sand of its delta. The course of the Lesse River in Belgium can be followed through underground caverns for a long distance in the famous Grott de Han. Many instances of the phenomena of the appearance and disappearance of rivers can be cited.

In regions where climatic, surface, and underground conditions are favorable the ground-water supply may be replenished during seasons of relatively high surface flow by flooding the ground surface and allowing the water to percolate into the ground.[1]

TABLE 27.—PROPOSED CLASSIFICATION OF SPRINGS ACCORDING TO RATE OF DISCHARGE

Order of magnitude..........	1	2	3	4
Average rate of discharge....	100+ sec.-ft.	10 to 100 sec.-ft.	1 to 10 sec.-ft.	100 to 448 g.p.m.

Order of magnitude.......	5	6	7	8
Average rate of discharge....	10 to 100 g.p.m.	1 to 10 g.p.m.	1/8 to 1 g p.m.	Less than 1 pint per minute

70. Velocity of Underground Flow. Velocity has components of both speed and direction. These are both measured by the use of dyes, soluble salts, and by electrical methods. In determining the direction of flow, dyes or salts are most suitable. The tracing reagent is placed in a central well at equal distance from which test wells have been sunk. The direction of flow is the direction from the central well to the well in which the reagent was first detected.

Fluorescein, uranine, and eosine are the dyes best suited to the purpose. Powdered fluorescein has a reddish-brown color; when dissolved in water it appears, by reflected light, a brilliant green. One part in 40 million can be detected by the unaided eye; 1 part in 10 billion can be detected by colorimetric analysis in long glass tubes. The dye has the advantage that it is unaffected by clays and will travel a long distance without change of properties.

Sodium chloride is used by placing it in the central well and making chemical analyses to determine the chloride content of the water in wells to which it is suspected the chloride is flowing. The method is not altogether satisfactory because, in long distances, the salt solution, whose specific gravity is higher than that of fresh water, sinks rapidly and may not reach the down-grade points and also because of the diffusion of the salt solution.[1]

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[1] See ROPER, R. M., "Ground Water Replenishment by Surface Water Diffusion," *Jour. A.W.W.A.*, February, 1939, p. 165.

The electrical method is used mainly for the determination of the speed of flow of underground water.[2] The electrical method is operated as follows:

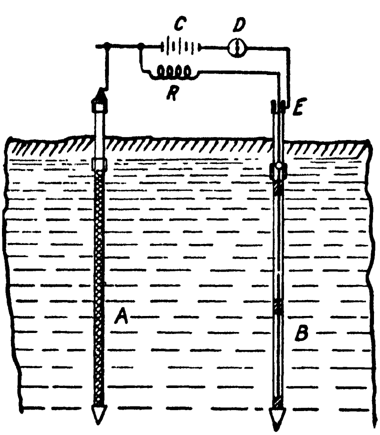

FIG. 32.—Diagram illustrating Slichter's method for determining the velocity of flow of underground water.

In Fig. 32, *A* is the casing of the upper well which acts as an electrode, *B* is an electrode in the lower well insulated from the well casing, and *C* and *D*, respectively, are a battery and an ammeter. *E* is a connection to the casing of the lower well. When connected for the test, a feeble current may be shown by the ammeter. A strong solution of an electrolyte is placed in the upper well, and the time is noted. Periodic observations of the ammeter and of the time are made and plotted. The appearance of the graph will be somewhat as shown in Fig. 33. In this figure the distance *AB* represents the time of passage of the ground water from the upper to the lower well. The sudden deflection at *B* shows the final arrival of the electrolyte at the lower well. The point *B* should be taken at the point of inflection of the curve and not at the highest, or

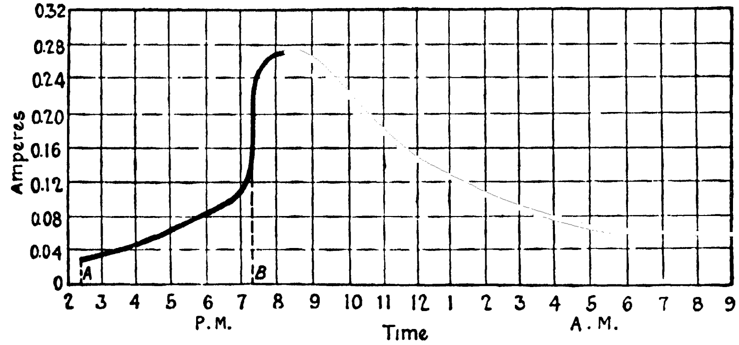

FIG. 33.—Curve obtained by Slichter's electrical method for determining the velocity of flow of underground water.

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[1] See also WENZEL, L. K., *U.S. Geol. Survey, Water Supply Paper* 679-A, p. 7, 1936.

[2] See also MEINZER, O. E., *Jour. A.W.W.A.*, April, 1935, p. 479; and FIEDLER, A. G., *U.S. Geol. Survey, Water Supply Paper* 596-A, 1928.

maximum, point. If the point of inflection is taken, the effect of diffusion of the electrolyte will be nullified.

71. Types of Wells. Underground water is usually brought to the surface by springs or wells. The following may be included among the various types of wells in use for water-supply sources:

Gravity Well. A gravity well is a vertical, or approximately vertical, hole from the surface of the earth penetrating saturated underground material in which the surface of the water outside of and surrounding the well is at atmospheric pressure.

Pressure Well. A pressure well, sometimes called an artesian well, is a vertical, or approximately vertical, hole from the surface of the earth, passing through an impervious stratum to penetrate an aquifer holding water under pressure greater than atmospheric. Hence, when the hole, or well, penetrates the aquifer, water tends to rise in the hole If the pressure is sufficient, the water will rise to the ground surface and the well will flow. It is to be noted that a flowing well is an artesian well, but an artesian well is not, necessarily, a flowing well.

Gallery. A gallery is a horizontal, or approximately horizontal, tunnel or open ditch, through water-bearing material. The tunnel type of gallery is sometimes called a horizontal well. Conditions of flow into galleries are usually such that the surface of the underground water outside the gallery is exposed to atmospheric pressure.

72. Well Problems. Among the engineering problems involved in connection with a well after its completion is the conduct of tests on which predictions of the characteristics and behavior of the well can be based; or it may be desired to specify the dimensions of a well to give desired characteristics if constructed in known underground conditions. Such tests and predictions are based mainly on mathematical formulas, partly rational, partly empirical, giving only approximately correct results. The development of these formulas is based on an assumption of the conditions of underground flow.

73. Conditions of Flow into Gravity Wells. Hypothetical underground conditions surrounding a gravity well, in which the entire length of the well exposed to the aquifer will admit water into the well, are illustrated in Fig. 34. If a test hole is bored into the ground at any point in the same aquifer as the gravity well, before water is withdrawn from the well, the water in the bore hole and in the well stand at the line *AD*, marked "original water surface," which is approximately horizontal. After water has been pumped from the well for some time, water will stand in the test bore holes, driven in a vertical plane passing through the axis of the well, at elevations along the line *AB'C'D*, marked "free-surface curve," provided the bore holes were only just deep enough to reach the free surface. It is to be expected that moisture would probably be found in the ground above the "original water surface" and above the free surface, due to capillarity, retention, or other forces. The original water surface and the free surface mark only elevations at which water would be encountered in the well and in the bore holes.

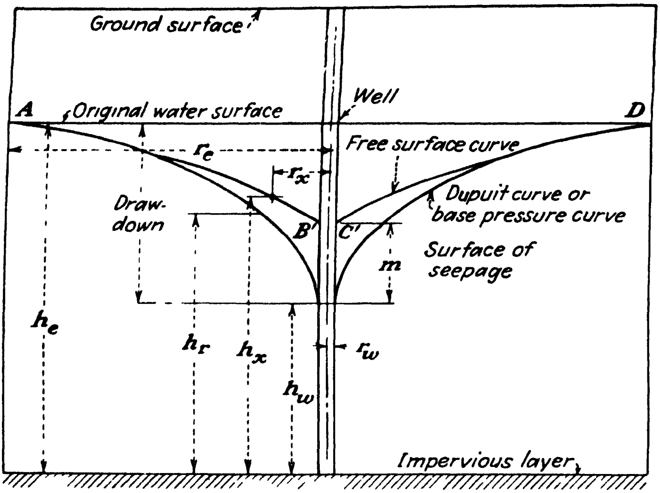

FIG. 34.—Underground conditions in gravity well.

The free surface marks the boundaries of the "cone of depression." This is the space in the aquifer from which water has been removed. The radius of the base of this cone, r, in Fig. 34, is known as the radius of the circle of influence. This radius is a constant only when the rate of withdrawal from the well is equal to the rate of replenishment, unless the circle of influence reaches a fixed limit. This condition is illustrated in Fig. 35. The distance from the original water surface to the surface of water in the well is known as the *drawdown*. When replenishment of the ground water surrounding a well is at a rate less than the rate of withdrawing water from the well, the drawdown will increase with time.

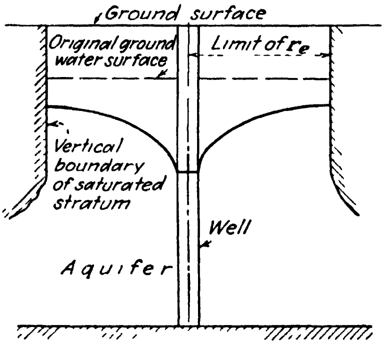
Ground surface
Original ground water surface
Limit of r_e
Vertical boundary of saturated stratum
Well
Aquifer

FIG. 35.—Well with limited boundary.

It is to be noted in Fig. 34 that the elevation of the water surface in the well while it is being pumped lies at some distance below the free surface outside and immediately adjacent to the well. The vertical surface, or face, of the ground forming the outside of the well hole that is exposed between the surface of water in the well and the free surface is known as the *surface of seepage*.

74. Physical Laws of Underground Flow. The flow of water through porous underground mediums is usually so slow, merely a few feet per day, that it occurs under conditions of stream-line, or nonturbulent, flow in which the head loss due to friction varies directly as the velocity of flow.

TABLE 28.—VELOCITY OF FLOW OF WATER IN SANDS OF VARIOUS EFFECTIVE SIZES OF GRAINS AND THE MAXIMUM FLOW OR TRANSMISSION CONSTANT FOR EACH SAND[1]

(Porosity, 32 per cent temperature, 50°F.)

Diameter of soil grain, millimeter	Velocity in feet per day for a pressure gradient of		Kind of soil	Diameter of soil grain, millimeters	Velocity in feet per day for a pressure gradient of		Kind of soil
	1 : 1	100 ft. per mile			1 : 1	100 ft. per mile	
0.01	0.163	0.0038	Silt	0.25	102	2 31	Medium sand
0.04	2 62	0.0605		0.45	331	7.47	
0.05	4.08	0.0922	Very fine sand	0.50	408	9.21	Coarse sand
0.09	13.2	0.298		0.95	1,470	33.2	
0.10	16.3	0.369	Fine sand	1.00	1,630	36 8	Fine gravel
0.20	65.4	2.09		5.00	40,800	921	

[1] SLICHTER, *U.S. Geol. Survey, Water Supply Papers* 67, p. 27, and 140, p. 1.

Under these conditions Darcy's law[1] controls, and it can be stated that

$$V = ks \tag{1}$$

and

$$Q = kAsp \tag{2}$$

where V = velocity, ft. per day
s = slope of hydraulic grade line, *i.e.*, slope of ground-water table
k = a constant, determined by conditions of flow and units used
A = cross-sectional area of aquifer
p = permeability of water-bearing medium

Hazen,[2] in his study of the flow of water through slow sand filters, concluded that

$$V = ke^2s\left(\frac{t + 10}{60}\right) \tag{3}$$

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[1] "Les Fontaines publiques de la ville de Dijon," 1856.

[2] HAZEN, ALLEN, *Mass. State Board of Health Rept.*, 1892, pp. 541, 553.

TABLE 29.—VELOCITIES OF FLOW OF UNDERGROUND WATER AS COMPUTED FROM HAZEN'S FORMULA
(Feet per day)
Effective size of sand in millimeters

Porosity	Very fine	Fine	Medium	Coarse	Very coarse		Fine gravel					Multiplier for values in Table 30
	0.10	0.20	0.30	0.40	0.50	0.60	0.80	1.00	1.50	2.00	3.00	
0.25	23.6	94.3	212	378	590	850	1,510	2,360	5,310	9,430	21,200	0.72
0.27[1]	32.8	131	295	525	820	1,181	2,100	3,281	7,380	13,100	29,500	1.00
0.30	46.6	186	419	745	1,160	1,680	2,980	4,660	10,500	18,600	41,900	1.42
0.35	77.4	309	696	1,240	1,935	2,790	4,960	7,740	17,400	30,900	69,600	2.36
0.40	128	486	1,092	1,950	3,040	4,380	7,780	12,200	27,400	48,600	109,200	3.71

[1] The values in this line are taken from Table 30.

where t = temperature, °F.

e = Hazen's effective size of sand[1]

Values of V are given in Tables 28, 29, and 30, for a temperature of 50°F., and for various values of the effective size. Hazen states that the formula is not applicable to gravel of sizes greater than about 3.0 mm.

75. Capacity of a Gravity Well. Under the conditions of flow illustrated in Fig. 34, characteristics or dimensions that are measurable in the field are h_e, h_w, r_w, and Q, where Q is the rate of pumping from the well. Characteristics affecting flow that cannot be measured under most field conditions include the porosity and perviousness of the aquifer, the size of the particles composing it, and the magnitude of r_e.

An approximate formula has been developed, involving all the measurable terms, in addition to the term r_e, by which it is possible to obtain approximate solutions to problems in connection with the flow of water into or from gravity wells. This is known as Dupuit's formula.[2] It was devised by Jules Dupuit, a French philosopher. His formula is based on assumptions of the conditions of underground flow that aproximate true conditions; therefore the results given by the formula may not be exact. Experience has shown that under most conditions the results given by the formula are of practical value. The formula is developed as follows:

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[1] Hazen's *effective size of sand* is defined as the size of grain in a sample of sand such that 10 per cent, by weight, of the sample has grains smaller than this size. The effective size is usually expressed in millimeters. To express, approximately, the distribution of grain sizes in sand Hazen defined the *uniformity coefficient* as the ratio of the size of grain in the sample such that 60 per cent, by weight, of the sand has smaller grains, to the effective size.

[2] DUPUIT, J., "Etudes theoriques et pratiques sur le mouvement des eaux," Dunod, Paris, 1863.

TABLE 30.—VELOCITIES OF FLOW OF UNDERGROUND WATER[1]
(The hydraulic grade line has a slope of 1:1, the temperature is 50°F., and the porosity in Slichter's formula is taken at 32 per cent)

Effective size of sand, millimeter	Slichter		Hazen		Effective size of sand, millimeters	Slichter		Hazen	
	Inches per minute	Feet per day	Meters per day	Feet per day		Inches per minute	Feet per day	Meters per day	Feet per day
0.01	0.0014	0.16	0.1	0.328	0.35	1.67	200	122 5	402
0.02	0.0054	0.65	0.4	1.312	0.40	2.18	261	160	525
0.03	0.0122	1.46	0.9	2.953	0.45	2.76	331	202.5	665
0.04	0.0218	2.62	1.6	5.25	0.50	3.40	408	250	820
0.05	0.0330	4.08	2.5	8.20	0.55	4.12	507	302.5	992
0.06	0.0490	5.88	3.6	11.8	0.60	4.90	588	360	1,181
0.07	0.0667	8.00	4.9	16.1	0.65	5.75	690	422 5	1,385
0.08	0.0871	10.5	6.4	21.0	0.70	6.67	800	490	1,608
0.09	0.1103	13.2	8.1	26.6	0.75	7.66	919	562.5	1,845
0.10	0.1361	16.3	10.0	32.808	0.80	8.71	1,046	640	2,100
0.12	0.1961	23.5	14.4	47.2	0.85	9.84	1,180	722.5	2,370
0.14	0.2668	32.0	19.6	64.3	0.90	11.0	1,324	810	2,660
0.15	0.3063	36.8	22.5	73.8	0.95	12.3	1,470	902.5	2,960
0.16	0.3485	41.8	25.6	84.0	1.00	13.6	1,630	1,000	3,281
0.18	0.4412	52.9	32.4	106	2.00	54.5	6,535	4,000	13,124
0.20	0.5446	65.4	40	131	3 00	122	14,700	9,000	29,527
0.25	0.8509	102	62.5	205	4.00	218	26,136		
0.30	1.225	147	90	295	5.00	340	40,800		
Values of k in the expression $v = kd.^2$...	13.61	1,633	1,000	3,281	....	13.61	1,633	1,000	3,281

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[1] The velocities determined by Hazen and Slichter are not comparable due to differences in conditions of the experiments from which their respective formulas were determined and also to differences in the determination of effective size.

The rate of flow of underground water, as shown in formula (2), is

$$Q = kAsp$$

It is assumed that the direction of flow of ground water is horizontal and that it flows radially toward the center of the well shown in Fig. 34; then

$$A = 2\pi r_x h_x$$

and

$$Q = 2\pi k r_x h_x sp$$

but

$$s = \frac{dh_x}{dr_x}$$

hence

$$Q = 2\pi k r_x h_x p \frac{dh_x}{dr_x}$$

or

$$Q \frac{dr_x}{r_x} = 2\pi k p h_x \, dh_x$$

integrating

$$Q \log_e r_x = \pi k p h_x^2 + C$$

when $r_x = r_e$,

$$h_x = h_e$$

whence

$$C = Q \log_e r_e - \pi k p h_e^2$$

and

$$Q \log_e r_x = \pi k p h_x^2 + Q \log_e r_e - \pi k p h_e^2$$

transposing

$$Q = \pi k p \frac{h_e^2 - h_x^2}{\log_e (r_e/r_x)}$$

but, when $r_x = r_w$

$$h_x = h_w$$

then

$$Q = \pi k p \frac{h_e^2 - h_w^2}{\log_e (r_e/r_w)}$$

If $K = \dfrac{\pi p k 7.5}{2.3 \times 1{,}440}$; values of k, in feet per day, are taken from Tables 28, 29, and 30; and Q is expressed in gallons per minute; then

$$Q = K \frac{h_e^2 - h_w^2}{\log (r_e/r_w)} \qquad (4)$$

The curve, or graph, resulting from the plotting of Dupuit's formula [formula (4)] produces the "pressure" curve, line *ABCD* in Fig. 34. It does not coincide with the actual underground water surface, that is the free surface. The difference between the two curves is a measure of the approximation of the results given by the use of Dupuit's formula. It has been found in practice that the approximations involved in the use of Dupuit's formula generally give results of practical value. The greater approximations occur in gravity wells where the drawdown is a large proportion of the well depth.

76. The Free-surface Curve. The *free surface* of the water in the cone of depression surrounding a well that is being pumped is the surface of the water that is under atmospheric pressure. It is the surface at which underground water would be encountered in a bore hole or test well adjacent to a well being pumped. It is to be noted that it does not coincide with Dupuit's "base pressure curve."

The equation of the free surface has been expressed as[1]

$$Q = \frac{\pi k h_e (h_e - h_x)}{2.3 C_x \log (r_e/0.1 h_e)} \tag{5}$$

in which values of k, in feet per day, may be taken from Tables 28, 29, and 30, C_x is a constant, to be taken from Fig. 36, and other nomenclature is shown in Fig. 34.

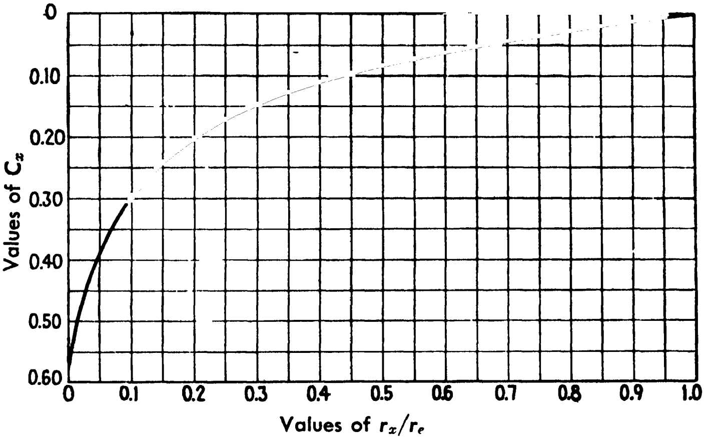

FIG. 36 Values of C_x to be used in Eq. (5)

77. Relation between Q, $h_e - h_w$, and r_e. Dupuit's formula can be rewritten as

$$Q = k \frac{(h_e - h_w)(h_e + h_w)}{\log_e (r_e/r_w)} \tag{6}$$

In order that the value of r_e may be a constant, the area of replenishment for horizontal underground flow must be large enough so that the rate of replenishment is equal to Q. Hence it may be assumed that

$$r_e = CQ \tag{7}$$

It is evident that where $(h_e - h_w)$ is small compared with $(h_e + h_w)$ the value of Q varies approximately as $(h_e - h_w)$, since the values of $\log r_e$ vary but slightly with values of Q. The straight-line relationship between Q and $(h_e - h_w)$ is more exact for pressure wells, because of the form of Eq. (13), page 80.

78. Partial Penetration. If a well does not penetrate to the bottom of the aquifer, the formula for full penetration is not applicable. Kozeny[2] has found that the expression of flow into a partially penetrating well is

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[1] From BABBITT, H. E., and D. H. CALDWELL, "The Free Surface around, and the Interference between, Gravity Wells," *Univ. Ill., Eng. Expt. Sta., Bull.* 374, 1948.

[2] KOZENY, J., *Wasserkraft u. Wasserwirtschaft*, Vol. 28, p. 101, 1933.

$$Q = \frac{k(h_e^2 - h_w^2)}{\log (r_e/r_w)} \left(1 + 7\sqrt{\frac{r_w}{2h_e}} \cos \frac{\pi P}{2}\right) \tag{8}$$

where $P = h_e/h$

h = thickness of aquifer, $h_e \lesseqgtr h$

In using this expression the penetration must be known. It may be found by test borings or from logs of adjacent wells. If the penetration is small, even a rough estimate of it introduces but little error into the corrected value of Q since for small values of P the term $\cos \pi P/2$ is very nearly equal to unity.

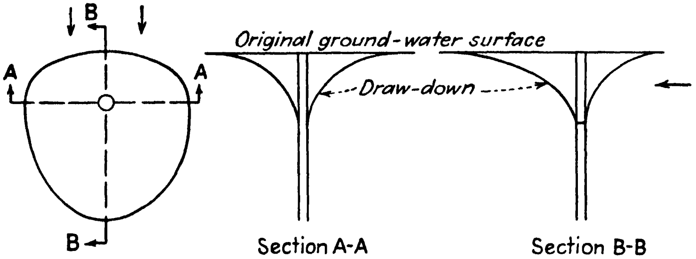

FIG. 37.—Conditions when well is not in middle of field of influence.

79. Area of Influence. The area of influence surrounding a gravity well in a sloping water table is not necessarily circular, and the well is not in the "center" of the area, as indicated in Fig. 37. It has been shown by Muskat[1] that for an appreciably noncircular area the expression

$$Q = \frac{\pi k(h_e^2 - h_w^2)}{2.3 \log_e (C/r_w)} \tag{9}$$

may be used to compute the flow into such a well, using for C an approximate average of the distances from the well to the boundary. The effect of the displacement of the well from the "center" of the area of influence may be neglected for displacements up to 50 per cent of the value of r_e.

80. Drawdown and Recovery Curves. The drawdown curve may be defined as a curve representing the decline of the water level in a well during pumping. A recovery curve represents the recovery of the water level in a well after pumping has stopped. The coordinates of both curves are time and drawdown. A typical drawdown-recovery curve is based on the application of the expression[2]

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[1] MUSKAT, M., "Flow of Homogeneous Fluids through Porous Media," McGraw-Hill Book Company, Inc., N.Y., 1937.

[2] CALDWELL, D. H., "The Hydraulics of Gravity Flow of Water Wells," Thesis, University of Illinois, 1943.

$$T = \frac{0.184 r_e^2 f}{Q}\left(h_e - \sqrt{h_e^2 - 1.7\frac{Q}{k}}\right) \tag{10}$$

in which T is time, in hours, to develop the radius r_e; f is the specific yield of the material, *i.e.*, the ratio of the amount of fluid that will drain from a saturated material to the volume of the material; and Q is the flow, in cubic feet per second; and on the expression

$$d_0 = \frac{1}{2.3}\log\frac{r_0}{r_w}\left(h_e - \sqrt{h_e^2 - 1.7\frac{Q}{k}}\right) \tag{11}$$

in which d_0 is the drawdown in the well, in feet, and r_0 is the radius of the circle of influence, in feet, corresponding to this value of d_0.

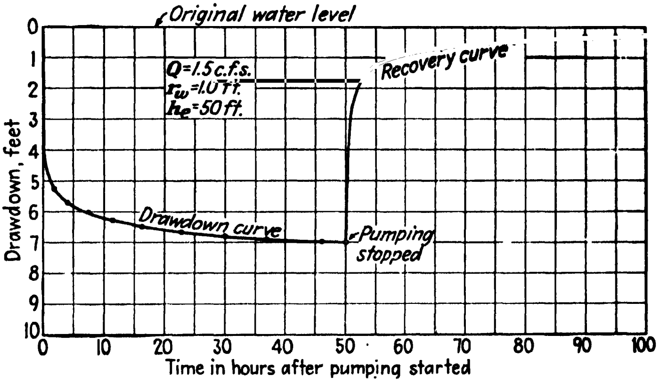

FIG. 38.— Drawdown-recovery curve. (*From D. H. Caldwell, "The Hydraulics of Gravity Flow of Water Wells," Thesis, University of Illinois, 1943.*)

The drawdown of the base pressure curve at any other point, expressed as d_0', may be determined by replacing r_w by r_x, where r_x is the distance from the well center to the point in question. If this distance is approximately equal to or greater than the thickness of the aquifer, the drawdown will be very nearly equal to the drawdown of the free surface.

81. Dupuit's Formula for Flow into a Gallery. The hypothetical conditions of flow into a gallery, or a line of wells closely spaced, are illustrated in Fig. 39. Following the form of the derivation of Dupuit's formula for flow into a gravity well and starting with the expression

$$Q = kAsp$$

the value of A, for a unit length of one side of the gallery, is found to be h_x. Substituting and integrating, as before, it is found that the rate of flow per unit length of one side of the gallery is

$$Q = k\frac{h_e^2 - h_w^2}{2r_e} \tag{12}$$

82. Dupuit's Formula for Flow into a Pressure Well. The hypothetical conditions of flow into a pressure, or artesian, well are illustrated in Fig. 40. Following the form of the derivation of Dupuit's formula for flow into a gravity well and starting with the expression

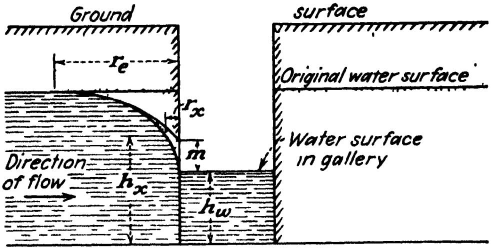

Fig. 39.—Flow into one side of a gallery.

$$Q = kAsp$$

the value of A is found to be $2\pi r_x t$. Substituting and integrating, as before, it is found that the rate of flow into a pressure, or artesian, well is

$$Q = k\frac{h_e - h_w}{\log_e (r_e/r_w)} \tag{13}$$

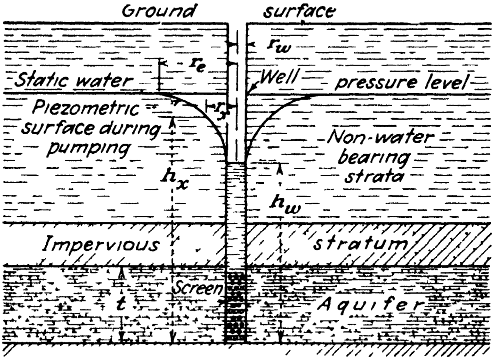

Fig. 40. Underground conditions of flow into a pressure or artesian well

83. Rate of Recovery in Pressure Well. The rate of recovery is so much faster in a pressure well than in a gravity well that it may be used as one of the easily observed characteristics to identify the type of a well. The rate of recovery in the pressure well is faster than in the gravity well because the water does not fill the cone of depression around the pressure well.

To express the rate of recovery in a pressure well Slichter[1] devised the expression

$$T = K \log \frac{y_0}{y_1} \tag{14}$$

where T = time for water level to rise from y_0 to y_1
K = a constant for the well and units used
y_0 = drawdown in the well at any time after pumping has stopped
y_1 = drawdown in the well at a time T after observation of the depth y_0

This is an approximate expression only, as it is based on the assumption that the radius of the circle of influence remains constant during the period of recovery. For the relatively small drawdowns in artesian wells, as compared with the depth of the well, the solutions obtained by the use of the formula are of practical value It is undesirable to use the formula where the drawdown is a large proportion of the total depth of the well.

84. Interference between Wells. If two or more wells are located so that their cones of depression intersect, they are said to interfere; the amount of interference in any well being represented by the ratio of the dimunition of flow from the well to the total flow from the well without interference. Muskat[2] has proposed formulas for the computation of interference effects that are based on logical analysis. Tests made on a laboratory scale at the University of Illinois[3] have indicated that the results from Muskat's formulas are in close agreement with field observations. Some of Muskat's formulas are as follows:

For two interfering wells, a distance W apart

$$Q_1 = Q_2 = \frac{K(h_e^2 - h_w^2)}{\log (r_e^2/r_u W)} \tag{15}$$

For three wells in a straight line, a distance w apart

Outer wells

$$Q_1 = Q_3 = \frac{K(h_e^2 - h_w^2) \log (W/r_u)}{2 \log (R/W) \log (W/r_w) + \log (W/2r_w) \log (R/W)} \tag{16}$$

Middle well

$$Q_2 = \frac{K(h_e^2 - h_w^2) \log (W/2r_w)}{2 \log (R/W) \log (W/r_w) + \log (W/2r_w) \log (R/r_w)} \tag{17}$$

where R = effective radius of influence

The conditions of interference between pressure (artesian) wells has been expressed by Balch,[1] based on work by Slichter, as

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[1] *U.S. Geol. Survey, Water Supply Paper* 10, 1905.

[2] MUSKAT, M , *loc. cit.*

[3] BABBITT, and CALDWELL, *loc cit.*

$$Q_2 = k \frac{h_e - h_w}{\log\,(2lr_w^3 + l^2r_w^2) - \log\,[r_e^2(l + r_e)^2]} \tag{18}$$

$$Q_3 = k \frac{h_e - h_w}{\log\,[(r_e^2 + l^2)r_e] - \log\,(l^2r_w)} \tag{19}$$

where Q_2 = flow into one of a pair of wells l distance apart
Q_3 = flow into a single well in a group of three or more wells equidistant l from each other

85. Examples. *Problem* 1. A gravity-type well is 24 in. in diameter. When it is being pumped at 500 g.p.m., the drawdown in a well 50 ft. away is 10 ft., and in another well 100 ft. away it is 3 ft. The well penetrates the aquifer, giving a depth of water in the well, when not being pumped, of 100 ft. (*a*) What is the drawdown in the well? (*b*) What is the specific capacity of the well? (*c*) What is the maximum rate at which water can be pumped from the well? (*d*) What would be the drawdown for a discharge of 300 g.p.m.? Solve by the use of Dupuit's formula and the use of the free-surface curve, and discuss the comparative results.

1. Solve for r_e. $Q = k\dfrac{h_e^2 - h_x^2}{\log\,(r_e/r_x)} = k\dfrac{100^2 - 90^2}{\log\,(r_e/_{50})} = k\dfrac{100^2 - 97^2}{\log\,(r_e/100)}$

Therefore

$$10 \times 190(\log r_e - 2) = 3 \times 197(\log r_e - 1.7) \quad \text{and} \quad r_e = 137 \text{ ft.}$$

2. Solve for C in the expression $R = CQ$. $C = {}^{137}/_{500} = 0.274$.

3. Solve for k. $500 = k\dfrac{10 \times 190}{2.136 - 1.7} = k\dfrac{3 \times 197}{2.136 - 2.0}$

$$k = \frac{500 \times 0.436}{1{,}900} = \frac{500 \times 0.136}{(3)(197)} = 0.115$$

4. Solve part (*a*) by substitution in Dupuit's formula

$$500 = 0.115\frac{100^2 - h_x^2}{\log\,({}^{137}/_{1})}; \qquad h_x = 26; \qquad \text{Drawdown} = 100 - 26 = 74 \text{ ft.}$$

5. For part (*b*), similarly, $Q = 0.115\dfrac{199}{\log\,(0.274Q/1)}$

By trial and error $Q = 27$ g.p.m.

6. For part (*c*), similarly, $Q = 0.115\dfrac{100^2}{\log\,(0.274Q/1)}$

By trial and error $Q = 535$ g.p.m.

7. For part (*d*), similarly,

$$300 = 0.115\frac{100^2 - h_x^2}{\log\,(82.2/1)}; \qquad h_x = 71 \text{ ft.} \qquad \text{Drawdown} = 100 - 71 = 29 \text{ ft.}$$

Now solve by the use of the free-surface formula.

1. Find value of r_e by trial and error. Assume $r_e = 150$ ft. Then when $r_x/r_e = {}^{50}/_{150} = 0.33$, C_x (Fig. 36) $= 0.14$; and when $r_x/r_e = {}^{100}/_{150} = 0.67$, C_x (Fig. 36) $= 0.05$.

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[1] See *Jour. A.W.W.A.*, June, 1930, p. 727.

Substituting in formula (5)

$$\frac{\pi k(100)(100 - 90)}{2.3 \times 0.14 \log (^{150}/_{10})} = \frac{\pi k(100)(100 - 97)}{2.3 \times 0.05 \log (^{150}/_{10})} \quad \text{and} \quad 0.50 = 0.42$$

Assume $r_e = 133$ ft. Then when $r_x/r_e = 0.385$, $C_x = 0.1153$, and when $r_x/r_e = 0.770$, $C_x = 0.033$.

Substituting in the formula, 0.35 = 0.36, a satisfactory result.

Conclusion $r_e = 133$ ft.

2. Find value of k.

$$500 = \frac{\pi k 100 \times 10}{2.3 \times 0.12 \log (^{133}/_{10})} = \frac{\pi k 100 \times 3}{2.3 \times 0.035 \times 1.124} \quad \text{and} \quad k = 0.049$$

3. To solve part (a): find C_x when $r_u/r_e = {}^{1}/_{133} = 0.0075$. Then $C_x = 0.525$. Substituting in the formula

$$500 = \frac{\pi \times 0.049 \times 100(h_e - h_x)}{2.3 \times 0.525 \times 1.125} \quad \text{and} \quad (h_e - h_x) = 43.8 \text{ ft.}$$

4. To solve part (b): Note that discharge varies directly as drawdown $(h_e - h_u)$ Hence, $(h_e - h_u) = 500/43\,8 = 11.4$ g.p.m.

It is to be noted that this "drawdown" refers to the surface of the water in the ground immediately adjacent to the well, and not to the surface of the water in the well.

5. To solve part (c): The equation of the free surface gives insufficient information to permit its use in solving this problem. A hypothetical solution can be obtained by Dupuit's formula.

6. To solve part (d): Proceed as in the solution of part (b).

$$h_e - h_x = \frac{300}{11.4} = 26.3 \text{ ft.}$$

Problem 2. A gravity-type well is 12 in. in diameter and discharges 150 g.p.m. with a drawdown of 10 ft. and 500 g.p.m. with a drawdown of 50 ft. The static depth of water in the well is 150 ft. What will be the discharge from the well for a drawdown of 20 ft.?

Solution: By Dupuit's formula,

$$150 = K \frac{10 \times 290}{\log \left(\frac{150C}{0.5}\right)} \qquad 500 = K \frac{50 \times 250}{\log \left(\frac{500C}{0.5}\right)}$$

Solving

$$C = 0.12$$

$$K = \frac{500 \log 120}{12{,}500} = 0.085$$

Then

$$Q = 0.085 \frac{20 \times 280}{\log (0.12Q/0.5)}$$

and by trial and error

$$Q = 264 \text{ g.p.m.}$$

CHAPTER V

LOCATION, CONSTRUCTION, AND MAINTENANCE OF WELLS

86. Location of Wells. In a region where the ground-water table is high and the underground strata are pervious, shallow wells of moderate capacity may be located at almost any convenient site. Such conditions exist in most regions of plentiful rainfall. However, the location of a deep, high-capacity well, such as is required for a public water supply, may require either exploratory prospecting or the application of the principles of underground flow, as described in Sec. 74, to a knowledge of the geology of the region and to a study of the characteristics of existing wells in the field, or to test holes sunk in the field. By exploratory prospecting is meant the drilling and test pumping of test wells to determine the direction of underground flow and possibly the limits of the field of underground water.

Wells may be successfully located where a study of geologic conditions indicates the presence of saturated strata. Valleys of surface or buried rivers offer the most promising location for wells, but underground water may be found on a mountain where there is still higher land to serve as a watershed and pervious strata to hold the water. Surface indications seldom give any inkling of the presence of artesian water, so that an artesian well must be located as a result of investigation or of previous knowledge of local conditions.

The existence of water in porous material at or above the level of a near-by surface stream that flows through the material can usually be depended upon. The farther the point is from the surface stream, the higher the elevation at which water will probably be found. The principal difficulty in locating a well arises from the location of the porous stratum. The porous stratum may rise and fall in an unexpected and inexplicable fashion; there may be faults in the substrata so that a dry well may be located close to a well giving a good yield. When a well is located near a stream, the water will enter the well from the surrounding ground and not from the stream, unless the radius of the circle of influence is large enough to include the surface of the stream.

Where the information on geology and existing wells is meager, helpful information may be obtained through scientific instruments applied in underground surveys and in geophysical prospecting.[1] Such prospecting[2] depends on the speed and manner in which underground strata of different characteristics convey sound waves or electric currents.

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[1] See also KELLY, S. F., "Engineering Use of Geophysics," *Civil Eng*, October, 1932, p. 628.

[2] See also BRUCKSHAW, J. McG., "Geophysical Prospecting and Water Supply Problems," *Water and Water Eng.*, September, 1946, p. 507.

In the seismic method of geophysical prospecting, sound waves, or stress waves, are propagated at or slightly below the ground surface by means of an explosion, usually of a few sticks of dynamite. These waves spread through the earth from the explosion as a center. The greater the density of the underground strata, the greater the speed of the waves. Some waves are reflected when encountering denser strata and return to the surface where sensitive instruments, attuned to select certain reflected waves, supply a record from which profiles of underground strata can be drawn. These profiles may be useful in indicating the location of water-bearing strata.

The resistivity method of geophysical prospecting depends on the resistance of underground strata to the flow of an electric current through them. The presence of water in the ground may reduce its resistance. A study of the electrical currents flowing underground may indicate the location of water-bearing strata.

Relatively little experience with geophysical prospecting is reported in technical publications in the waterworks field, most of the practical applications having been made in the search for oil.

Care should be taken not to locate wells in a region where the quality of the water will be endangered by surface drainage or by the existence of drainage wells used for underground disposal of waste waters or in a region in which the top of the well may be flooded by impure surface water. All wells, particularly shallow dug wells, should be located at a safe distance from sources of pollution. In homogeneous materials, such as sand or gravel, a minimum distance of 250 ft. is recommended between the well and a source of pollution, such, for example, as a privy vault. In some cases 100 ft. is permitted. Local conditions must control. No fixed distance for all situations can be assuredly *safe*.

87. Grouping of Wells. Grouping of numerous wells in a well field depends on the type of aquifer from which the water is drawn, the area of land under the control of the waterworks, the rate at which the wells are to be pumped, and possibly other conditions. In order to obtain the greatest amount of water available from a limited area in which the wells penetrate an underground reservoir from which water may be drawn from all directions, some interference between wells is necessary. The minimum amount of interference that is economical is fixed by the cost of piping to bring the water from the wells to a central point and by the cost of power transmission to the wells. Interior wells in a group that penetrates an underground reservoir should be further apart than exterior wells in the group because of the greater effect of interference between such interior wells. A number of wells penetrating an underground stream with distinct channel and direction of flow should be placed in a line transverse to the stream and near enough together to minimize costs and, at the same time, avoid excessive interference. Successive rows of wells can be placed downstream to increase the volume of water available from the underground stream, but the specific capacity of such wells will decrease downstream when upstream wells are being pumped. In order to avoid uneconomical interference, the distance between rows of wells downstream should increase progressively. The economical spacing between wells in any field and group can be determined only by experience in that field.

In general, wells pumped by suction should be close together, and the suction line should be free from turns and obstructions to minimize head losses, since the available suction head is usually low. Leakage of air into the suction line is difficult to avoid. This air, combined with gases released from solution by the lowered pressure, causes trouble in drawing water from the wells.

When locating wells near property lines, it should be borne in mind that if water is taken from beneath neighboring property to the injury of its owner, damages can be recovered therefor. The waterworks administration should be able to demonstrate that the circle of influence of the waterworks wells could not have passed beyond the waterworks property.

88. Legal Restrictions on Well Construction. Four states, Wisconsin, Montana, Utah, and New York, have laws affecting the construction of wells, and the trend seems to be toward further legal safeguards of well construction. The New York law refers only to counties on Long Island. The other states require the licensing of well drillers and require that certain information be furnished concerning the construction of the well. Legal restrictions on the use of ground water seem necessary to conserve the diminishing supply in many areas.[1] Legal bases for such restrictions are outlined in Sec. 7.

89. Types of Well Construction. Wells may be classified according to the method of construction as dug, bored, driven, jetted, or drilled. The first three are commonly confined to relatively shallow wells not ordinarily used for large water supplies; although in a few instances large dug wells[2] have been constructed for large users. Some methods for the protection of small wells against surface contamination are illustrated in Fig. 41. The jetting process has been used in sinking both shallow and deep wells, but for the sinking of the deepest wells drilling is used almost exclusively.

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[1] See also TOLMAN, C. F., and A. C. STIPP, "Analysis of Legal Concepts of Subflow and Percolating Filters," *Trans A.S.C.E.*, Vol. 106, p. 882, 1941.

[2] See COE, C. B., "Large Water Supply Well Sunk by Open Caisson Method," *Eng. News-Record*, Vol. 79, p. 460, 1917.

Artesian wells are almost all drilled wells. Care should be observed in their construction not to penetrate into the artesian aquifer until the hole has been cased and grouted to prevent the loss of water into other strata.

Tentative specifications for the construction of deep wells have been adopted by the American Water Works Association.[1] Four of the eleven types of wells described in the specifications are illustrated in Figs. 42 and 55. It is generally recommended that the method of sinking a well be left to the contractor unless the writer of the specifications knows that a particular method will give superior results.

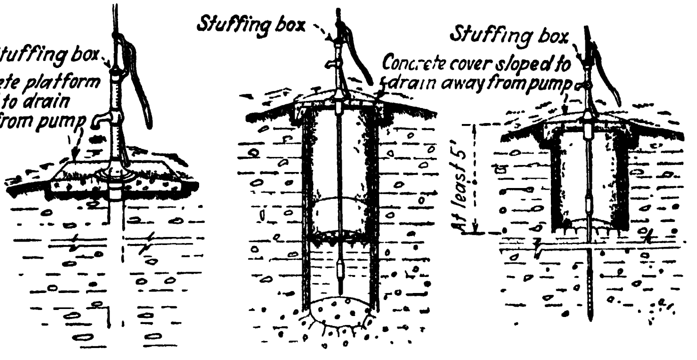
Stuffing box
Concrete platform sloped to drain away from pump
Stuffing box
Stuffing box
Concrete cover sloped to drain away from pump
At least 5'

Fig. 41.—Methods of well construction. Recommended by the U.S. Public Health Service. (*Public Health Report Reprint* 952, 1924.)

90. Dug, Bored, and Driven Wells. Dug wells are commonly excavated by hand tools, the excavator descending into the well as the excavation progresses. Where the walls of the well will not stand without support they are lined, or "curbed," usually with a watertight lining near the surface to exclude polluted surface water and with a pervious lining near the bottom to admit ground water. Dug wells are commonly constructed 3 to 4 ft. in diameter and to depths of 30 to 100 ft., although deeper dug wells are in existence.

Wells are sometimes constructed by driving, or pulling, an open-end casing into the ground and removing the material by boring, drilling, sand bucket or bailer, orange-peel bucket, or other means, depending, to some extent, on the diameter of the bore hole. In such wells the casing is either driven down with hammer blows; pulled down by jacks attached to deeply buried anchors or deadmen, or to other objects; or allowed to sink of its own weight. In the last method, excavation proceeds slightly in advance of the casing. The method is known as the caisson method. It is used for holes from 4 to 6 ft. and up to 50 or more feet in diameter.

The Ranney water collector is a dug well about 13 ft. in diameter that

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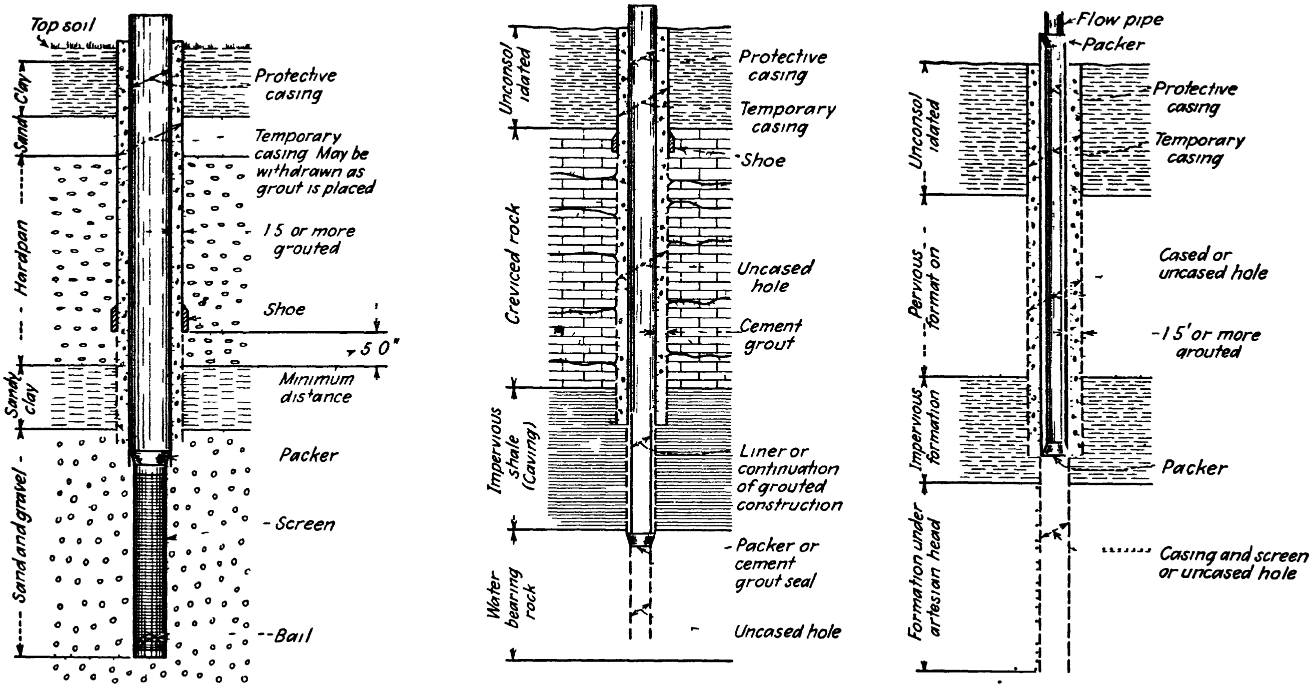
[1] See *Jour. A.W.W.A*, September, 1945, p. 913.



UNCONSOLIDATED MATERIALS

CONSOLIDATED MATERIALS

FIG 42 Standard types of well construction (From *Specifications for deep wells* ' *Jour A W W A September* 1945 *p* 913)

has been sunk as a caisson. Screens are driven radially and approximately horizontally from this well, as indicated in Fig. 43.[1] The length of these horizontal screens varies from 100 to 300 ft., lengths up to 200 ft. being most common. Such wells have great capacities, some up to 25 m.g.d.[2]

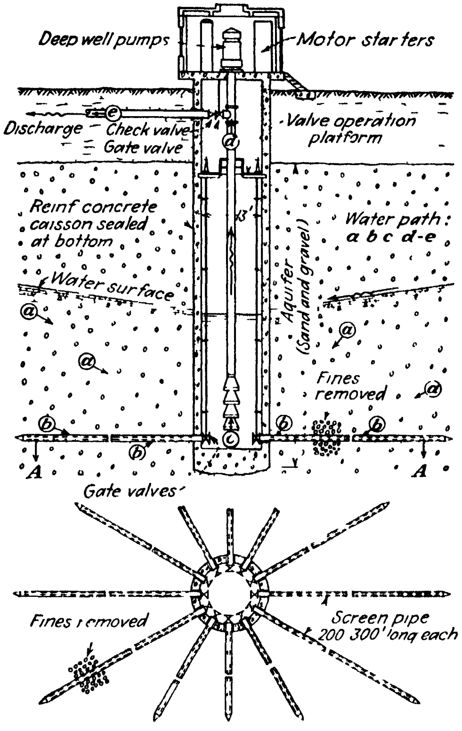


FIG. 43—Radial well or Ranney water collector (*From R. Nebolsine, Jour. New Engl. Water Works Assoc., Vol 57, p* 191, 1943.)

91. Drilled Wells. The successful sinking of deep drilled wells requires special training, experience, tools, and equipment. Among the various methods of drilling wells may be included (1) the standard method, with or without modifications; (2) the jetting method; (3) the core-drill method; and (4) the hydraulic rotary method.

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[1] "The Ranney System of Underground Water Collection," *Engineering* (British), Dec. 12, 1941, p. 461.

[2] See NEBOLSINE, ROSS, "New Trends in Ground Water Development," *Jour. New Engl. Water Works Assoc.*, Vol. 57, p 191, 1943; and KAZMANN, R. G., "Induced Filtration Supplies Most Productive Well Field," *Civil Eng.*, December, 1946, p. 544.

92. The Standard Method. The standard method involves percussion drilling, in which a drill is alternately raised and dropped in the descending bore hole. It is suited to the drilling of wells in any material from soft clay to the hardest rock. Other methods, such as the hydraulic rotary and the core-drill methods, may advance more rapidly through hard rock. Modifications of the standard method include the "pole-tool" method, in which wooden rods are used instead of rope; and the "hollow-rod" method, in which the string of tools above the boring bit is supported by hollow rods through which water is drawn upward to remove the drill cuttings continuously.

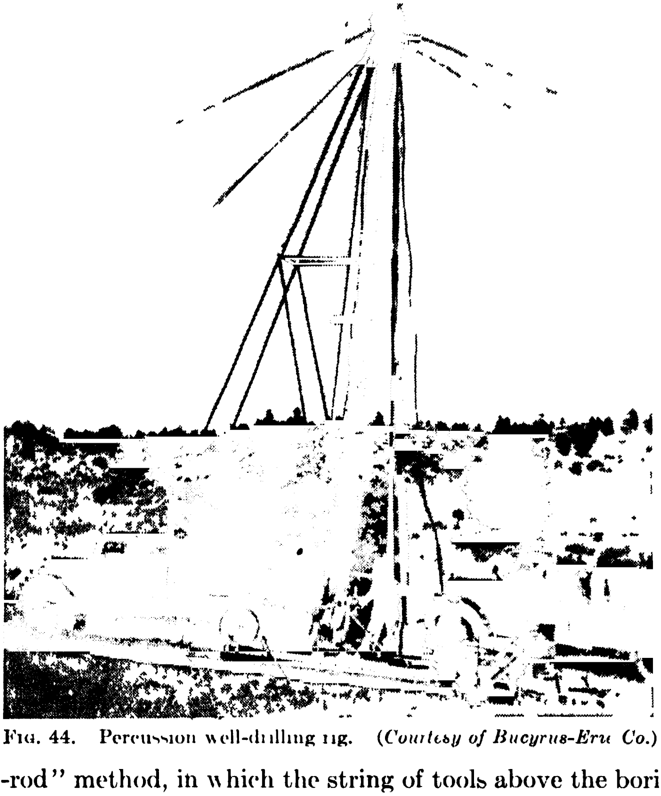

FIG. 44. Percussion well-drilling rig. (*Courtesy of Bucyrus-Erie Co.*)

A light percussion drill rig adapted to the standard method is illustrated in Fig. 44, and a set of tools and accessories used with the rig is illustrated in Fig. 45.

The procedure in drilling with such a rig is as follows:[1] (1) The rig is raised, and a mud sill is placed under it to give the mast a solid foundation.

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[1] Taken mainly from *War Dept. Tech. Manual*, TM5-295, 1942.

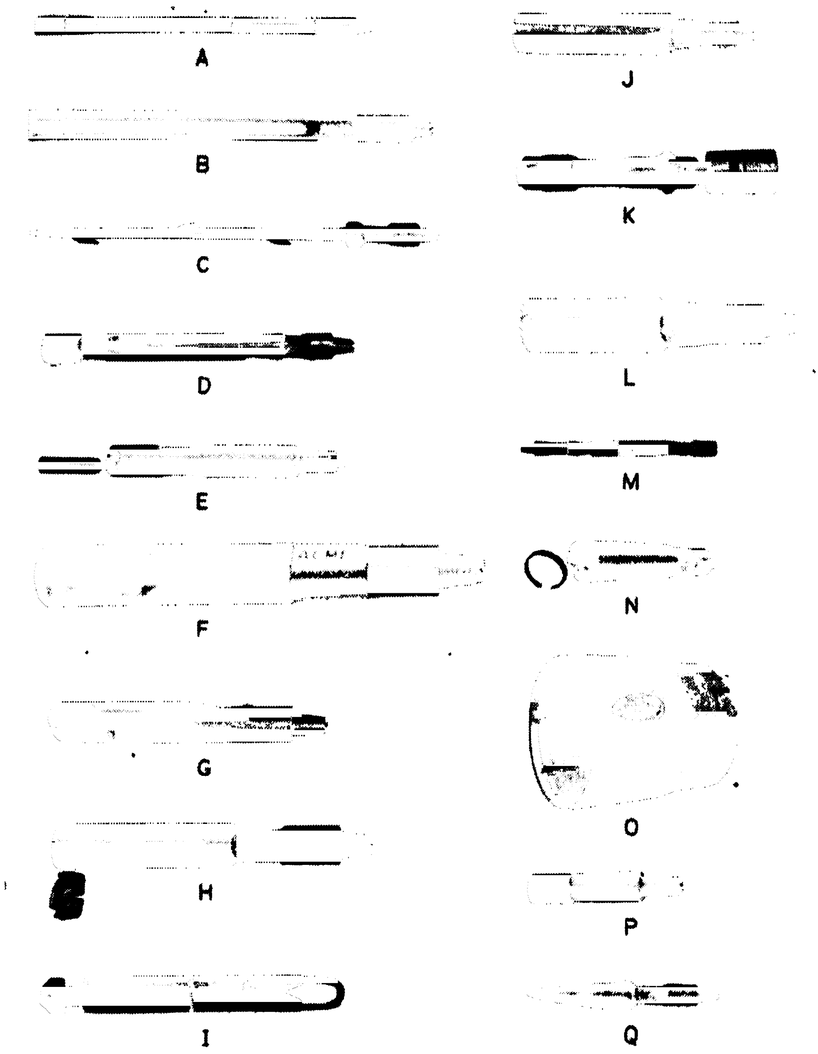

FIG. 45.—Well-drilling tools. (*Courtesy of Acme Fishing Tool Company.*)

A. Drilling stem
B. Drilling bit
C. Fishing spear
D. Drilling jar
E. Prosser swivel socket
F. Casing cutter
G. Collar buster
H. Side-slot combination socket and socket slips
I. Bailer
J. Full-circle slip socket
K. Hollow spear
L. Drill mill
M. Fishing jar
N Ring-collar socket
O. Die nipple
P. Rope socket
Q. Mandrel or swage

Jacks are placed between the rig frame and mud sill to level the rig and take the weight off the springs. Guy lines from the mast are lead to deadmen or other holdfasts. (2) A wooden platform is built at the rear of the rig, surrounding the hole that is to be drilled. The platform should be at a height to make the control levers accessible to the driller. A sluice box and drain are built to receive and carry off bit cuttings and water as they are bailed from the well. (3) The string of tools, as illustrated in Fig. 46, is assembled and inserted in the drill-stem guide. The latter may be used in starting the hole and until its depth is equal to the length of the drill bit. However, it may be better to start the hole with an earth auger. As the hole deepens, the string of tools may be lengthened by increasing the length of the auger stem, to aid in keeping the hole straight, and by the addition of "jars." The jars are a loose link which gives a sharp upward blow to lessen the tendency of the drill bit to stick. (4) The rig is started, and the bit is alternately raised and dropped to crush, ream, and penetrate the strata. Water is run into the hole during drilling to carry off some excavated material and to facilitate the bailing of the hole. When the bit has penetrated a short distance, it is withdrawn from the hole, and the bailer, operated on a separate cable, is lowered into the hole to remove the cuttings.

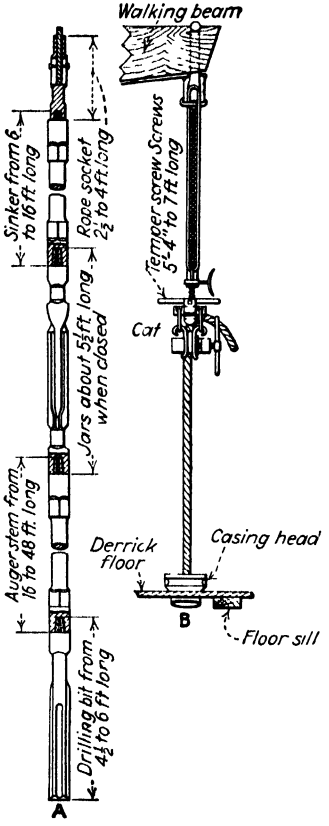
Walking beam
Sinker from 6 to 16 ft long
Rope socket 2½ to 4 ft long
Temper screw Screws 5'-4" to 7 ft long
Jars about 5½ ft long when closed
Cat
Auger stem from 16 to 48 ft long
Derrick floor
Casing head
B
Floor sill
Drilling bit from 4½ to 6 ft long
A

Fig. 46.—*A*, string of tools used with standard drilling outfit; *B*, temper screw. (*From U.S. Geol. Survey, Water Supply Paper* 257, 1911.)

At the start of the hole the full weight of the drilling tools is allowed to strike the ground to be penetrated. When the penetration is sufficient, the stretch of the rope is used to aid in causing the tools to rise after the blow has been delivered by the drill.

If the hole is to be cased, the first length of casing, to the lower end of which a drive shoe may be attached, is lowered into the hole when it reaches a sufficient depth. The drilling continues inside the casing, lengths of casing being added as progress is made. Drilling inside the casing is possible since the bit drills a hole 1 to 2 in. in diameter larger than the body of the drill inside the casing.

93. Jetting Methods. In this method the tools consist of a hollow drill, drill pipe, and water swivel. The equipment includes a derrick from which the pipe is supported, a force pump, and necessary water and power. Casing and a drive weight must also be provided. The hole is advanced by pumping water under pressure through the drill bit while it is churned up and down. The casing follows the bit very closely, and the water rising through the casing lifts the finely divided cuttings from the well.

This method of drilling is suitable in soft, unconsolidated, alluvial deposits and for wells of one diameter for depths up to 400 to 500 ft. If a greater depth is desired, the diameter of the hole must be decreased as the depth of the bore increases. In some materials it is not necessary to case the well until the hole is completed. In suitable materials a well can be sunk by this method faster than by any other. There are records of wells 500 ft. deep which have been completed ready for service in two days.

94. Core-drill Method. Core drills consist of a hollow bit armed with abrasive teeth on the annular circumference of the bit. The bit is rotated; it is not advanced by percussion. The parts of a core drill are illustrated in Fig. 47. As the drill advances in solid material, a core rises in the center. This is broken off, raised, and should be preserved to furnish a record of the material penetrated.

A diamond drill is a core drill. The diamonds used are either carbons or borts. As the drill revolves the diamonds cut into or abrade the rock, the cuttings being removed by water as in the hydraulic rotary method. A rotary-core drill with steel teeth can also be used with success.

Another method of advancing a bore hole by a rotary abrasive method is by the use of steel shot poured into the hole and held beneath a revolving annular steel shoe. This is known as the "chilled shot" method.

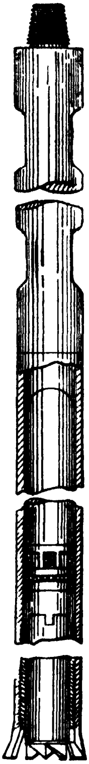

FIG. 47.—Core drill. (*From U.S. Geol. Survey, Water Supply Paper* 257, 1911.)

95. Hydraulic Rotary Drilling. In the hydraulic rotary method of drilling,[1] a mud-laden fluid is pumped from a sump at the ground surface, down to the bottom of the well through the hollow drill stem and through a hole into the cutting bit. The fluid returns up the well, carrying with it the cuttings as they are loosened by the revolving drill bit. The actual drilling tool consists of a fishtail, disk, or special rock bit shown in Fig. 48 and Fig. 49. These are connected to a hollow drill stem, made up in 20-ft. lengths which are connected at the top to a square rod, known as the "Kelly," which receives its rotary motion from a table similar to that shown in Fig. 50.

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[1] TIMMINS, L. P., "Mud-flush Rotary Drilling for Water Wells," *Water and Water Eng.*, Apr. 19, 1930, p. 161.

Among the advantages of this method of drilling are the speed by which the hole can be advanced, particularly in soft materials, and the plastering effect of the mud-laden fluid as it rises through the bore hole. The hydraulic head thus developed aids in preventing the caving of the walls of the hole before the casing has been set and simplifies the setting of the screen and casing in the hole before completion of the well. The rotary core barrel makes it possible to determine the characteristics of the formations being drilled, and the use of the rock bit now makes it possible to advance through hard rock without the use of a combination of the rotary- and cable-tool outfits.

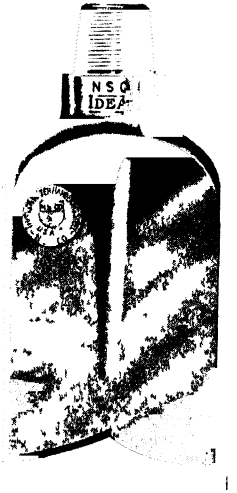

FIG. 48. Fishtail drill bit, showing hollow stem (*Courtesy of National Supply Company.*)

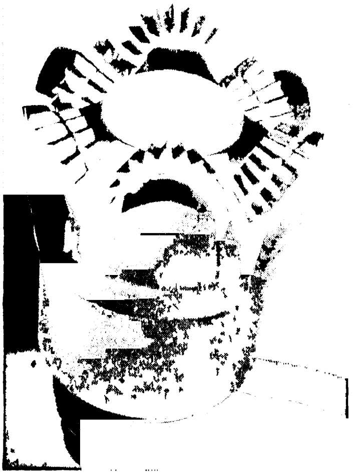

FIG. 49. Hard-rock cutter head (*Courtesy of Hughes Tool Co.*)

96. Difficulties in Drilling. Difficulties in well drilling are the result of the loss of tools, bad alignment of the bore hole, encountering of boulders, movement of plastic clay, encountering of quicksand, and other causes. Tools are lost through the breaking of the rope, which may be caused by defective material or by the jamming or "freezing" of the tools in the well. A bit may become wedged in the material being cut, as a result of striking too often in the same place, or tools may freeze because of rising sand or the sedimentation of cuttings if the pumps are stopped. Lost tools may be recovered by a number of ingenious devices, some of which are illustrated in Fig. 45. The position of the lost tool must be determined with accuracy by direct observation, by photography,[1] or by a wax impression. The fishing tool must be used with skill and care so as not to make conditions worse. The alignment of a well is maintained by the use of a long auger or drill stem.

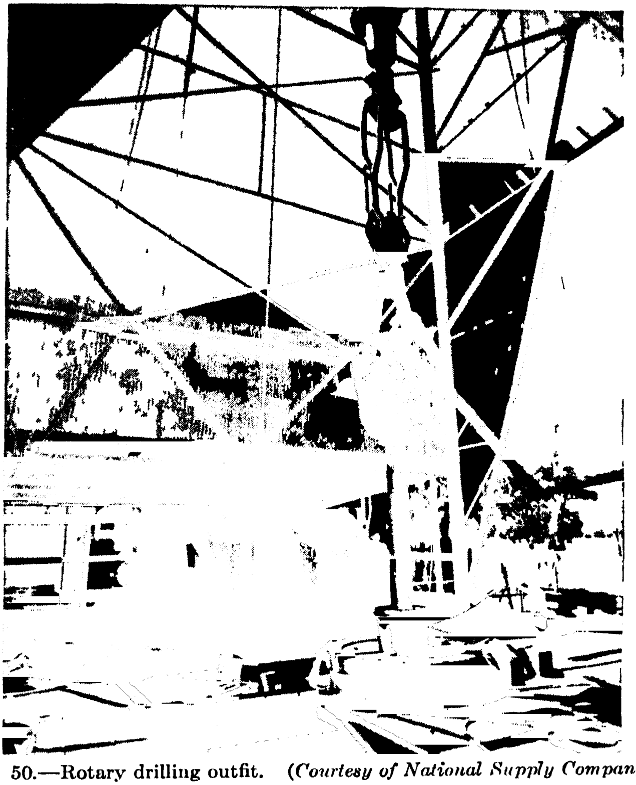

FIG 50.—Rotary drilling outfit. (*Courtesy of National Supply Company*)

97. Deviation from the Vertical. During construction, and before a well is accepted or the pumping equipment is installed, the plumbness and alignment should be tested. Relatively slight deviations from the vertical can be measured by lowering a plumb bob into the well and noting the distance between the plumb-bob string and the top of the casing as the bob strikes the side of the casing at various depths. If the deviation from the vertical is greater than the diameter of the casing, two methods are available for making the necessary observations. In one a glass bottle containing about a 50 per cent water dilution of hydrofluoric acid is lowered into the well and allowed to stand for about 15 min. to permit the acid to etch the glass. The line of the surface of the acid shows the angle of deviation from the vertical. Both the direction and the angle of deviation can be detected by lowering into the well a compass floating in liquefied gelatin. The gelatin solidifies and fixes the compass in position, and the surface of the gelatin shows the angle of deviation from the vertical. Photographic equipment is available that will record a log of the course of the well. The alignment of the well can be tested by lowering into it a "dummy" consisting of a length of pipe slightly smaller in diameter than that of the well casing. The length of the dummy should be such as to detect undesirable deviations from a straight line. In general, a well should not deviate more than half its diameter for each 100 ft. of length.[1]

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[1] "Borehole Surveying," *Water and Water Eng.*, March, 1935, p. 112.

98. Depth of Well. There is no relation between depth and capacity of a well. The well must be deep enough to penetrate the aquifer from which water is to be drawn, to permit the installation of an adequate length of screen and the installation of the desired pumping equipment. Water wells are seldom more than a few hundred feet deep, because at great depths water is less likely to be potable. Oil wells have been sunk to depths greater than three miles.

99. Diameter of Wells. The diameter to be chosen for a well is affected by the method of construction, the amount of water to be drawn from the well, and other factors. In practice, some correlation between diameter of deep drilled wells and capacity can be found, as indicated in Table 31. In deep drilled wells the minimum size of hole may be affected by the equipment necessary to reach the required depth. In general, the increase of the diameter of a well for the purpose of increasing its specific capacity is not economical. However, an increase in the diameter of the well screen may be justified in order to decrease the velocity of flow into the screen and thus increase the life of the well.

TABLE 31.—CORRELATION BETWEEN CAPACITY AND DIAMETER OF DRILLED WELLS

Capacity, g p.m.	Diameter, in.	Capacity, g p m.	Diameter, in.
50	6	1,000 1,500	12
50 300	8	1,500 2,400	15
300 1,000	10	2,400 3,000	20
		3,000 and over	24

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[1] IMBT, W. C., "Crooked Holes," *Ill. Well Driller*, February, 1933, p. 10.

Deep wells may decrease in diameter at various stages of increasing depth, mainly for convenience in sinking the well. However, wells to be pumped by air lift should be smaller in diameter at the bottom than at the top to allow for the increasing volume of air at the lower pressures near the top of the well.

100. Log of a Well. The log of a well is a record of the underground materials penetrated at various depths and of the diameter and other characteristics of the well. In most states the keeping of a log is required by law, the log being filed for record with the Geological Survey or a similar bureau.

The logging of existing wells whose capacity has diminished has sometimes been found economically justifiable in a study of the best procedure in redeveloping the well. Results of such logging may include the location of lost tools, broken casing, cave-ins, strata that are taking water away from the well, and misplaced or broken seals. The results of acidizing or blasting wells and of the placing of gravel may be studied. Water horizons, detected behind casings by logging the well, have been recovered by punching or shooting holes in the casing.

Geophysical methods of logging[1] are similar to those used in the location of wells but are more extensively developed. They include seismic, acoustic, chemical, electrical, mechanical, optical, thermal, and radioactive methods. Seismic and acoustic waves are similar, except that the latter are of higher frequency and are absorbed differently. The characteristics of such waves propagated at the surface of the ground are detected by suitable instruments at various depths in the well. Obstructions in, and other features of, a well may be located by the detection of echoes from them resulting from the firing of a blank cartridge at the top of the well.

Chemical methods involve the determination of the characteristics of the formations penetrated by the well. There is a variety of electrical methods depending, in the main, on the measurement of the resistivities of the underground strata. The magnetic method depends on the fact that most rocks will retain magnetic properties when placed in a magnetic field. The length of time of retention, called the "retentivity," gives a clue to the characteristics of the rock. Mechanical methods include hole calipering and, during construction, a record of drilling rate and drill reaction. Optical methods include the taking of photographs, the measurement of the intensity of light reflected from the walls of the well, or the measurement of fluorescence induced by irradiating the well, usually with ultraviolet light. Thermal methods involve the measurement of the temperature gradient down the well or the direct measurement of the heat conductivity of the rocks. Artificial radioactivity can be induced in the rocks by a bombardment of neutrons, even through the casing. The induced radiations can subsequently be picked up in the hole by suitable instruments. Mechanical and optical methods of logging water wells are most commonly used.

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[1] See also BAYS, C. A., and S. H. FOLK, "Geophysical Logging of Water Wells in Northeastern Illinois," *Jour. Western Soc. Engrs.*, September, 1944, p. 248.

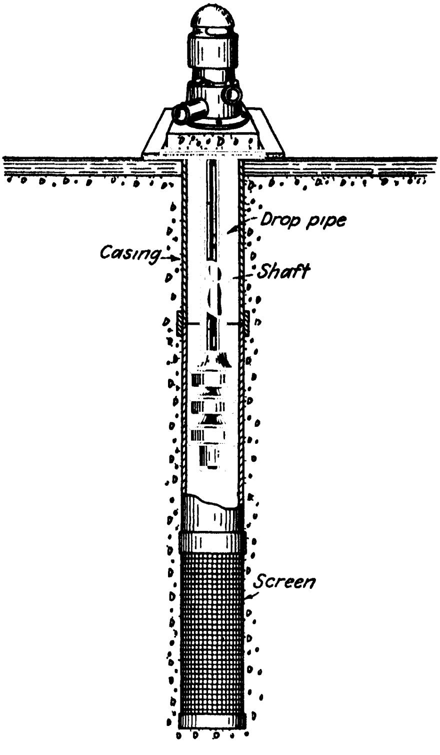

FIG. 51 Typical deep-well equipment.

101. Well Casing. The well casing, as shown in Fig. 51, forms the lining of the well. Near the top of wells of large diameter it may be called the "curbing." Among the purposes of well casing, or curbing, are the prevention of (1) the collapse of the hole, (2) undesirable water from entering the well, (3) good water from escaping from the well, and (4) material from falling into the well. It is frequently necessary to case wells for the entire length of the hole; in some instances, however, casing has been omitted through strata that will not crumble, such as dry solid rock.

Materials[1] most commonly used for well casing are wrought iron, alloyed or unalloyed steel, and ingot iron. The use of cast iron for the purpose is increasing. Cement-lined and enamel-lined pipes of the above materials are available and are suitable in some wells. In some instances wells have been cased with copper pipe, and others with asbestos-cement pipe, as well as fiber and vitrified clay.

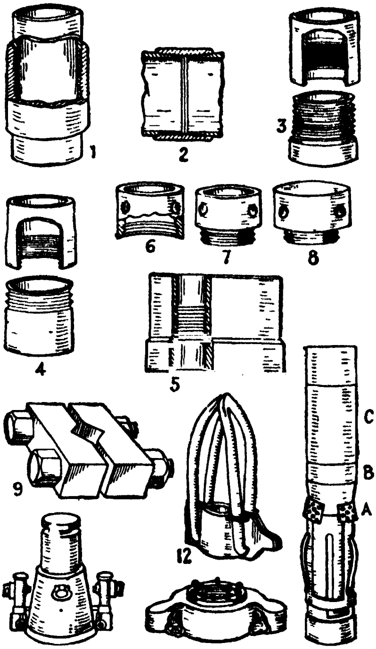

Fig. 52.—Well-casing couplings, clamps, and tools. 1, sleeve coupling; 2, tapered sleeve coupling; 3, 4, sleeve and inserted-joint couplings; 5, shoe; 6, 7, 8, drive heads; 9, drive clamps; 10, pipe ring; 11, hydraulic jack; 12, elevator; 13, gas packet. (*From U.S. Geol. Survey, Water Supply Paper* 257, 1911.)

Some types of casing are illustrated in Fig. 52. Where the casing is subject to considerable stress in driving or placing, alloys of iron, copper, and other metals are used to combine strength with resistance to corrosion. Soft alloys and lined pipes are unsuitable for casing that is to be driven. The weights of ferrous well casing are greater than for standard steel pipe.

Joints must be strong to withstand the stresses of driving and pulling. Some types of casing joints are illustrated in Fig. 52. Joints between casings of different diameters may be made tight by rubber or expanding lead gaskets.

Where metal casing is to be driven, the lowest section may consist of a sharpened tempered-steel shoe which cuts a hole large enough to be cleared by the remainder of the casing and the couplings. In soft materials the casing will follow the drill by its own weight, but if it sticks it must be driven down. A driving cap is placed on the top of the casing to protect it, and it is struck by a heavy wooden maul. If this is insufficient to advance the casing, drive clamps are bolted to the upper end of the auger stem. The machinery is then coupled as for spudding, and the drive cap is struck repeated blows by the driving clamp. If the pipe becomes badly stuck, it may be loosened by capping the top and forcing water under pressure into the well. As the water rises, it may carry the clogging material out with it.

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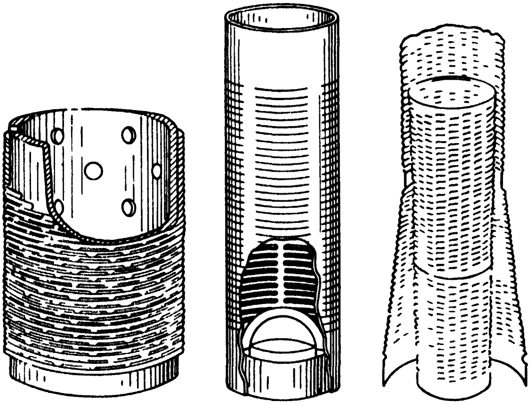
[1] See "Tentative Standard Specifications for Deep Wells," *Jour. A.W.W.A.*, September, 1945, pp. 913 and 944.

It is sometimes desirable to withdraw the drive pipe or casing for various purposes, as for the placing of a screen. Short lengths of casing are pulled by a hook attached to the rope run over the crown pulley of the derrick. Long lengths are pulled by lifting with hydraulic jacks which are fastened to a ring at the top of the pipe.

Well casings should, in general, not be used as part of the discharge pipe from the well, as possibilities of corrosion and loss of the well are increased.

102. Screens.[1] A screen is required at the bottom of a well to prevent the walls of the aquifer from caving into the hole, to exclude fine sand or granular material, and to permit the entrance of water. Some types of screens are illustrated in Fig. 53. A common method of placing a screen in a cased well is to drive the casing to the full depth of the well. The screen is then lowered inside the casing until it rests on the bottom of the well. The casing is then raised to expose the screen, and the joint between the screen and the casing is closed with a rubber packer or a lead packing ring which is expanded against the casing by a special tool.



LAYNE STRAINER COOK STRAINER LAYNE SHUTTER SCREEN

Fig. 53.—Types of well screens.

The size, shape, and spacing between openings in the screen are important conditions affecting the life of the well. The size of the openings may be determined, after a study of a mechanical analysis of the water-bearing stratum, to permit the passage of all fine particles representing a certain percentage, by weight, of the water-bearing material. It is not now uncommon to use openings that will pass 70 per cent, or more, of the sand grains in the aquifer, where the uniformity coefficient is high. The shape of the opening should prevent clogging and bridging.

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[1] See also Bennison, E. W., "Construction and Use of Well Screens," *Jour. A.W.W.A.*, June, 1939, p. 939.

These can be diminished by the use of openings that are larger on the inside of the well, as shown in Fig. 54. Any regular shape such as a rectangle, a square, a circle, or an oval can easily become clogged by sand or corrosion. Long, narrow, horizontal slotted openings, as illustrated in Fig. 53, are satisfactory. The openings should be placed as close together as structural strength will permit. This distance usually allows five to six openings per vertical inch. Screens formed by punching holes in the metal in order to leave a raised portion of the metal over the hole with a slot at the bottom, as shown in Fig. 53, are known as "shutter screens." They may be suitable as inside screens in a gravel-packed well, but because of the varying size and irregularity of openings they are not suitable in sandy material.

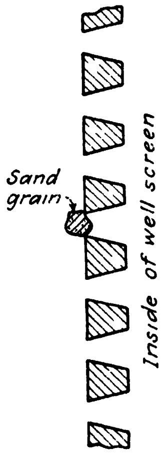

FIG. 54.—Section through wall of a well screen showing small opening on outside of screen compared to the larger opening on inside.

The total area of the openings in a screen should be such as to maintain an entrance velocity less than necessary to carry the finest particle of sand that is to be excluded by the screen. Lifting velocities are given in Table 32. The length of the screen should not be less than the thickness of the aquifer penetrated. The length, diameter, and total area are interdependent dimensions that must be adjusted to give the desired entering velocity. Some margin of safety in screen size is desirable to allow for incrustation and clogging and to prolong the life of the screen. The length of a screen should usually not be specified in advance of the construction of the well, in order that the length may be adjusted to the thickness of the aquifer penetrated.

TABLE 32.—LIFTING VELOCITIES OF WATER

Diameter of Grains, Millimeters	Velocity of Water, Feet per Second
Up to 0.25	0.0 to 0.10
0.25 to 0.50	0.12 to 0.22
0.50 to 1.00	0.25 to 0.33
1.00 to 2.00	0.37 to 0.56
2.00 to 4.00	0.60 to 2.60

Materials used for screens include iron or steel, either black or galvanized; corrosion-resistant alloys, some of which are listed in Table 33; concrete; wood; asbestos-cement; and gravel.

The material to be selected must have adequate structural strength and corrosion resistance, and it should be both available and not too costly. Since the useful life of the well is mainly dependent on the durability of the screen, emphasis is placed on the selection of the material of the screen.

To remove a screen from a well without collapsing the screen, the sand-joint method may be used. In this method sacking or similar material is fastened to the end of a pulling pipe that is lowered to the bottom of the screen. Sufficient clean sharp sand is then poured into the well almost to fill the screen. Where the space between the pulling pipe and the screen is large, the sacking must be supported by a truncated cone or disk welded to the pulling pipe. The joint between the screen and casing is broken, and the pulling force is transmitted uniformly through the sand to the screen, usually lifting it without damage.

TABLE 33.—SOME ALLOYS USED IN WELL SCREENS

"Maintaining Well Yields," *Water Works Eng.*, Aug. 27, 1941, p. 1061.

Alloy	Percentage of Each Constituent
Monel metal	Nickel 70, copper 30
Super nickel	Nickel 30, copper 70
Everdur metal	Copper 96, silicon 3, manganese 1
Stainless steel	Steel 74, chromium 18, nickel 8
Silicon red brass	Copper 83, silicon 1, zinc 16
Anaconda red brass	Copper 85, zinc 15
Armco iron[1]	Iron 99.84
Toncan iron[1]	Iron 99.55, copper 0.40, molybdenum 0.05

[1] Black or galvanized.

103. Corrosion of Screens.[1] Corrosion of well screens may be classified as direct, selective, and electrolytic. A uniform destruction of the surface, with enlarged openings, is typical of direct corrosion resulting from chemical reactions between the water and the screen. Selective corrosion is confined to alloys in which the corrosive water reacts mainly with one constituent of the alloy. Typical of this corrosion is dezincification and degraphitization. Electrolytic corrosion results from galvanic action set up by the presence of dissimilar metals in electrolytic solutions. This type of corrosion is discussed in Sec. 334. Direct and selective corrosion may be diminished by choosing other metals. Electrolytic corrosion may be avoided by using metals close to each other on the electrolytic scale.

104. Incrustation of Screens. Incrustation may be caused by the direct deposition of suspended matter from the water on to the screen, by the release of dissolved minerals from solution due either to the change in pressure at the screen or to chemical reactions, and to biological activities resulting in the deposition of gelatinous zoogloeal material on the screen. The principal incrustant is calcium carbonate, which both forms a scale on the screen and cements deposited particles. Carbon dioxide in solution under the high pressures in the aquifer may be released at the screen surface, resulting in the deposition of calcium carbonate. Sometimes aluminum silicates, iron, and sulphates are found in the incrustations.

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[1] *Ibid.*

Incrustation can be minimized by allowing a low head loss through the screen, thus preventing the escape of dissolved gases and the precipitation of incrustants. The removal of incrustations on screens is discussed in Sec. 117.

105. Gravel-wall Wells. Gravel-wall wells, with interior screens, as illustrated in Fig. 55, are used to increase the area of contact between the aquifer and the well construction, thus diminishing the velocity of the entering water. The gravel wall is placed during the construction of the well, various methods for placing the gravel being used. Among these methods are the following: (1) A drill bit and casing shoe, 4 to 6 in. larger than the outside of the casing, are used in drilling. This annular cylindrical hole is filled with gravel that sinks with the advancing casing. (2) The well may be constructed with an inner and an outer casing with diameters differing by 6 to 12 in., or more. When the proper depth has been reached, the screen is set, the inner casing is carefully centered, and the space between the two casings is filled with gravel. Sand may be pumped out through the lower open end of the inner casing, before it is sealed, so that additional gravel is pulled into the excavation, and conditions are left somewhat as indicated in Fig. 55. (3) If, in the preceding method, sufficient gravel cannot be put into the well, hydraulic jetting around the bottom of the casing will remove more sand, or the well hole may be underreamed by the use of expanding reamer blades. This should be done with water containing mud in suspension so that the sand walls will be held in position until the gravel has been placed. Care must be taken in the use of this method to avoid either caving or sealing of the sand stratum.

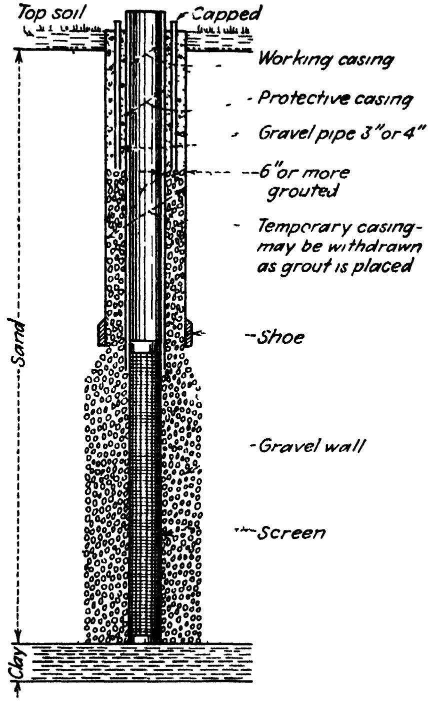

FIG. 55 Gravel-wall well (*From Jour. A.W.W.A., Vol 37, p 938, 1945*)

Gravel should be selected of a size that will prevent the entrance of particles of sand greater than is desired. For example, Gerber[1] states: "If the natural sand grains have a diameter of 0.04 in., the pack gravel could have a diameter of 0.25 in., or if the sand were very fine, say 0.005 in., two sizes of pack gravel might be used: 0.03 and 0.2 in., approximately." The openings in the inner metallic screen should be as large as possible without permitting the entrance of gravel into the well.

The cost of gravel-wall wells is usually greater than of typically screened wells, but the resulting increase in capacity and prolonged life frequently justify the increased cost.

106. Finishing a Well. After the screen has been set, as described in Sec. 102, or the casing has been perforated to admit water when no screen is to be placed, the well should be protected against contamination, as described in Sec. 107, and undesirable aquifers should be sealed off. The well is now ready for the installation of the drop pipe and pump, as illustrated in Figs. 51 and 126.

107. Protection from Contamination. Wells may be contaminated either by surface water entering at the top or following outside the casing, or by interchange of waters between water-bearing strata. Much attention has been given to the prevention of such contamination, and exhaustive specifications for the sanitary protection of wells have been published by the U.S. Public Health Service in a Sanitation Manual for Public Water Supplies.[2] Some items in the Manual are as follows:

Item 1. *Exclusion of Surface Water from Site.* The top of the well must be so constructed that no water from the well can drop back into it, the ground surface must slope away from the well or the site otherwise drained, and flood waters should not approach within 50 ft.

Item 2. *Earth Formations above Water-bearing Stratum.* Earth formations above aquifer should provide sufficient filtration to prevent contamination from surface water.

Item 3. *Distance to Source of Contamination.* The minimum permissible horizontal distance to a source of contamination is 50 ft., with 300 ft. preferred.

Item 4. *Depth of Casing and Curbings.* It must extend at least 10 ft. below the ground surface and preferably 10 ft. below the ground-water table.

Item 5. *Construction and Use of Casing and Curbings.* For drilled wells the space between casing and well hole is to be filled with not less than $1\frac{1}{2}$ in. of cement grout to a depth of 10 ft. or more. Casing is not to be used as suction pipe.

Item 6. *Gravel-treated Wells.* Gravel surface must terminate not less than 10 ft. below ground surface. Remaining space above gravel surface is to be filled with impervious clay, mortar, or cement grout. Gravel must have been disinfected before placing in the well.

Item 8. *Well Seals or Covers.* There must be a watertight seal or cover at the top of the casing. Recommended connections at the top between casing and drop pipe or suction pipe include: an all-flanged or threaded connection; metal to grouted cement with suitable gasket (not rubber) to be used only when the joint carries the weight of the pump and is rigidly supported to prevent vibration

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[1] GERBER, W. D., discussion, *Jour. A.W.W.A.*, April, 1937, p. 486.

[2] See *Jour. A.W.W.A.*, May, 1944, p. 501.

Item 9. *Well Vents.* These are necessary to retain atmospheric pressure in casing, preventing suction of contaminated water into the well. Such vents should extend above the floor bend and should terminate in a covered or screened return bend facing downward.

Item 10. *Well Pits.* Pumping machinery is not to be located below ground in pits.

Item 11. *Construction and Installation of Pumps* Among the details included are: pumps must not be primed by unsafe water; there must be a watertight connection between casing and power unit, or casing must extend 6 in or more above surface.

Item 12. *Pump House.* Adequately drained and protected against flooding.

108. Sealing a Well. Wells are sealed both during and after construction to prevent surface or underground contamination from entering the well, to prevent the passage of good water underground from one aquifer to another, to prevent the escape of gas from the well, and for other reasons. The sealing of the annular space near the ground surface is particularly important in creviced rock formations with connections to the ground surface.

A satisfactory method of sealing a well below the surface of the ground[1] requires that the outside casing be driven to the point at which the well is to be sealed. Drilling is continued below the casing for the full depth of the stratum that is to be sealed off, but in no case is the length of the hole below the casing to be less than the length of the string of tools. The hole is then filled with sand to the level at which the end of the inner casing is to be set, and the remainder of the hole is filled with neat cement, either dry or as a soupy grout. The inner casing, with a flat plate welded over its lower end, is lowered into the hole, pushing aside the grout until its lower end reaches the sand. The cement grout is thus pushed into the annular space between the casings. After the cement has set, the welded plate at the bottom of the inner casing is punched through, the sand is bailed out, and the drilling of the hole is continued. The sand is used so that the cement will not be cracked when drilling is resumed. A similar procedure may be followed in sealing an existing well.[2] Pressure cement grouting can also be used to seal out undesirable water.

It may sometimes be necessary to stabilize formations of a caving nature by injecting cement grout into them or to guard against well failure in such material by setting the lower terminal of the screen or casing in cement grout. In grouting the annular space surrounding the casing, the grout should be applied in one continuous process starting at the bottom of the space to be filled, where such depths are not greater than about 100 ft. The grout may be a mixture of 2 parts of sand to 1 of cement, containing not more than 5½ gal. of water per cubic foot.

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[1] CARR, J. A., "Experiences in Well Construction," *Water Works & Sewerage*, March, 1937, p. 94.

[2] See also HOLLAND, FRED, "Water Well Cementing," *Ill. Well Driller*, Vol. 16, No. 1, p. 7, 1947.

109. Chlorinating a Well.[1] Well tools are sometimes disinfected during construction, and all new wells should be chlorinated, as a sanitary precaution, before being turned into production. A well can be chlorinated by filling it with a solution containing at least 50 to 100 p.p.m. of available chlorine and allowing the solution to stand in the well about 24 hr. A stock solution of chlorine, containing about 15,000 p.p.m., may be applied at different levels after which the water in the well may be agitated to diffuse and to distribute the chlorine.

110. Performance Requirements. Specifications for the construction of a well sometimes include a statement of the required performance of the well under test conditions. The usual guarantee covers the quality and quantity of water to be delivered for a particular drawdown. Payment for the well may be in direct proportion to the specific capacity.

111. Well Tests.[2] New wells may be subjected to acceptance tests, and existing wells may be tested to determine their condition and capacity. Wells may be tested during construction to indicate whether they should be driven deeper or otherwise developed to attain the required capacity. The test usually requires the determination of the rate of flow from the well, *while the drawdown remains constant,* for a number of hours in the case of a gravity well. The maximum rate at which water can be drawn from a well for a short period of time, without regard to the drawdown, bears no relation to the long-time capacity of the well. Careful observation of the movement of the water level in a gravity well is of greater importance than in a pressure or artesian well because of the relatively slow recession of the water level in a gravity well penetrating highly pervious water-bearing material.

Where it is impractical to continue the test long enough to reach the desired drawdown, observations of rates of flow at lesser drawdowns may be made and used in the estimation of rate of flow at the required drawdown. In general, it is recommended that the water level in the well shall always be more than 5 ft. above the highest perforation on the screen.

112. Measurement of Water Level in a Well. The depth of water in a well while it is being pumped can be measured by any one of a number of methods. The equipment required in one method, involving the use of air pressure, is shown in Fig. 56. Before the pumping of the well is started, air is pumped into the air tube *D* until all the water has been expelled from the tube at *C*. As soon as air escapes at *C*, the pressure shown on the gage will cease to rise. The pressure on the gage will show the depth of water *AC* before pumping. As pumping continues, the water in the well will drop to some such position as *BC*. The maximum air pressure that can be developed on the pressure gage will be a measure of this depth *BC*. The drawdown will be the difference between *AC* and *BC*. By pumping air slowly and continuously into tube *D* and recording the change in air pressure on the gage, in terms of water depth in the well, a continuous record of the drawdown can be made.

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[1] See also "Sanitation Manual for Public Ground Water Supplies," *Pub. Health Reports* 59, No. 5; and *Jour. A.W.W.A.*, May, 1944, p. 501.

[2] See also "Tentative Standard Specifications for Deep Wells," *Jour. A.W.W.A.*, September, 1945, pp. 913 and 950.

In using the device shown at *E* a measured length of the rubber tube is lowered into the well until the level of water in the manometer changes. The length of tube let down the well, less the head *h* in the manometer, is the distance to water in the well.

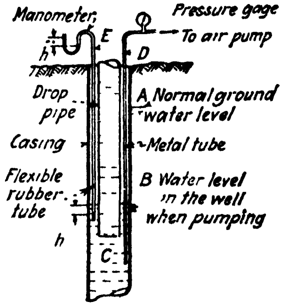

FIG. 56.– Air-pressure methods for measuring depth to water in a well.

Other methods for determining water level in a well involve the lowering of electric contact points into the well between the casing and the drop pipe. When the contact points touch water, the circuit is closed and a light comes on, a bell rings, or some other signal is given.

113. Maintenance of Wells. Among the causes of well failures that must be guarded against are (1) overpumping, (2) lowering of the water table, (3) clogging or collapse of screen, (4) leaky casing, (5) clogging of the sand or crevices in the aquifer, (6) leaky drop pipe, and (7) worn pump. All can be guarded against or detected by vigilance in operation. Overpumping is a frequent fault in operation. It results in clogging and incrustation of the screen, pumping of sand, and lowering of the water level in the well and lowering of the ground-water table surrounding the well. Pumping of sand may result also from collapse of the screen or the casing. To avoid these difficulties the maximum rate of pumping should be limited to that for which the well was constructed.

The lowering of the ground-water table by overdraft on the well field cannot always be avoided through the operation of a single well. As the water level is lowered, the specific capacity of the well is decreased and the power required is increased. It may be necessary to increase the length of the drop pipe in the well, to increase the depth of the well, or both.

Clogging of the screen may be detected by a drop of the water level in the well during pumping without proportionate fall in the level of the surrounding ground water. Clogging of the screen may result from sand lodging in the openings, from incrustation, and rarely, from bacterial action. Collapse of the screen may be indicated by the pumping of sand either with or without symptoms caused by clogging of the screen.

Leaky casings may admit undesirable water, the quality of which is detected by chemical analysis, or they may allow water to escape from the well into "thieving" strata. A leaky drop pipe and a worn pump will be indicated by a loss of pump capacity at known speeds of operation. "Thieving" strata may be detected by logging the well. Little can be done in the maintenance of a well to permit clogging or arching of the sand or clogging of the crevices in the aquifer other than to avoid overpumping. Leaky drop pipes and worn pumps can be observed by pulling them from the well periodically for examination at the surface.

114. Developing Wells. By the developing of a well is meant the restoration or increase of its capacity. An attempt to restore the failing capacity of a well should be made only after a study of the costs involved and the probable results. It may be less costly to construct a new well. A well may be developed by (1) removing, cleaning, and replacing the screen; (2) removing the screen and blasting, reaming, or otherwise reconstructing the well, and replacing the screen; or (3) without removing the screen, surging, acidizing, chlorinating, overpumping, or blasting and reconstructing. Where water-bearing aquifers that have been cased off are discovered, the casing may be perforated or cut opposite to them and a screen placed in proper position. In wells larger than 6 in. in diameter, or deeper than 100 ft., it is usually more economical to attempt to restore the well without removing the screen. Screens that have been removed may be cleaned on the surface of the ground with acid and by brushing, and repairs may be made to the screen.

115. Blasting Wells. The development of wells by blasting is most successful in consolidated material in which crevices are opened and interconnected by the shock of the blast. In unconsolidated materials sand grains may be more densely packed by the blast, resulting in a diminution of flow from the ground into the well.

Speed, high power, low bulk, and certain detonation are desired characteristics of the explosive to be used. Speed and power are required to overcome the tremendous pressures underground and to shatter the rock; a dense explosive is needed because of the small space available to hold it; and certainly of detonation is essential, as a misfire deep in a well hole cannot be safely rectified. The explosives that are most commonly used are 80 per cent high-velocity gelatin and nitroglycerin. Where small charges can be used, 40 per cent and 60 per cent dynamites are sometimes used. The sizes of shots vary from 50 to 200 lb. It is customary to begin with the lighter charges, progressively increasing them until the desired result has been obtained. The size of the charge is affected also by the water pressure to be overcome, the greater the pressure, the heavier the charge.[1]

The hole should be bailed after each shot, the size of the next shot being based on the character of the bailed-out material. Holes have sometimes been lost by overshooting.

116. Surging. To surge a well is to cause water to flow rapidly from the well into the ground, or alternately to overpump the well and to force water into the ground. The principal purpose of surging a well is to dislodge clogging and encrusting material from the screen. Immediately after a well has been surged, it should be strongly and continuously pumped until all dislodged material has been removed. Otherwise the improvement resulting from surging will be only temporary. Surging must be done carefully to avoid damage to the screen, and it is not always successful, occasionally causing the loss of the well.[2]

Reversal of flow of water into a well may be effected by discharging water from the distribution system, from the pumps, or from an elevated reservoir, under pressure, into the well. In surging with a plunger the drop pipe is removed from the well, and a solid plunger, fitting the walls of the casing, is lowered beneath the water in the well. The plunger is attached to a well rig and moved violently up and down, as in spudding, causing water to rush in and out of the well through the screen. By placing a check valve in the plunger, water can be forced through the screen in one direction only. It may serve as a convenient method for violently and briefly overpumping the well. If the top of the well casing is sealed, compressed air can be discharged into it to force water violently back through the screen. If air is permitted to follow through into the aquifer, it may cause "air logging" or clogging of the water-bearing sands with pockets of air. The sudden release of a large volume of compressed air into the well while the well is being pumped at the same time has been found effective in restoring yield.

Dry ice is solidified carbon dioxide. When dropped into a well, the solid quickly turns to a gas, generating a strong pressure if the gas is confined. In surging a well with dry ice, a charge of 50 to 500 lb. should be inserted and supported near the top of the casing, which is then quickly and tightly closed. Fast work is necessary owing to the low temperature of the carbon dioxide. The charge is suddenly released to fall into the well and vaporize. Pressures up to 90 p.s.i. have been generated in this manner. Although the method has been successful, some dangers accompany its use, such as frozen hands, suffocation due to dense fumes of carbon dioxide, rupture or lifting of the casing, or collapse of the screen.[1]

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[1] See also DOERR, K. P., "Use of Explosives in Blasting Water Wells," *Ill. Well Driller*, February, 1941, p. 8.

[2] See also HENDERSON, A. D. H., "Redeveloping Well Yield," *Water Works Eng.*, August, 1943, p. 882.

117. Acidizing and Chlorinating. Wells are acidized or chlorinated for the purpose of dissolving or dislodging clogging material or incrustation on the screen or in the sand around the screen. Acids may be used only where the metal of the screen will not be attacked by them. Brass or bronze may be treated with muriatic acid, HCl, while iron requires nitric acid.[2] Inhibited acids[3] contain chemicals that diminish the reaction between the acid and certain metals but not between the acid and the incrustants. Many inhibitors are proprietary compounds, some of which contain arsenic or other toxic substances that must be used with great caution if used at all. Stabilizers in acids retain in solution certain materials that would otherwise be reprecipitated as the acid weakens through dilution and neutralization. Acids should be applied in the highest concentration possible in the expectation that the acid concentration will reach at least 25 per cent in the screen. In some cases, progressively stronger doses of acid have been applied to the well. Acid is sometimes added by pouring it down a hose, with its lower end at the bottom of the well, and slowly raising the hose as the volume of the acid displaces the water in the well.

Muriatic acid is most commonly used as it is effective in dissolving calcium carbonate, the most common incrustant. The addition of about 3 per cent citric or acetic acid to the muriatic acid eliminates some trouble in iron-bearing water. Calcium sulphate incrustant may be dissolved by ammonium salts. Acid with a content of about 27 per cent muriatic acid is a commercial strength commonly used. If results are not satisfactory, the use of the muriatic acid may be followed by the application of sulphuric acid. The acid may be allowed to stand in the well for 1 to 24 hr., during which time the well may be gently surged intermittently for short periods of time. The well should be bailed clean of all dislodged material and the acid removed by flushing the well with water or neutralizing with lime, soda ash, or other chemical, followed by thorough flushing with water until traces of chemicals are removed.

Among the dangers to be guarded against in handling acids, besides acid burns, are the acid fumes, carbon dioxide which is generated in quantities large enough to be suffocating, or the release of explosive hydrogen.

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[1] See also Porter, R. H., "Well Reclaimed by Dry Ice," *Water Works Eng.*, Apr. 23, 1941, p. 446.

[2] See also Goetz, J. R., and F. L. Coventry, "Treating Encrusted Screens," *Water & Sewage Works*, April, 1946, p. R-23.

[3] See also Sec. 404.

Chlorine has been added to wells to remove incrustants resulting from the activities of the so-called iron bacteria. It serves also to produce hypochlorous acid, which will attack calcareous deposits.[1] In some cases it has been applied successfully ahead of acidizing.

118. Pumping Sand. Sand should be removed from well water before it passes through pumps or into the distribution system. Various types of sand-removing equipment are available, including the Brassert strainer[2] and a centrifugal device described by Jessup.[3] Bunker[4] describes a quick method for the detection of sand in well water based on visual comparison with the amount of sand in known standards. Sand may be pumped so rapidly from a well as to cause underground cavities which, extending to the surface, undermine the foundations of surface structures. There is no remedy for this after the damage has been done. A warning is given by the appearance of sand in water pumped from the well. The undermining of surface structures can then be avoided by pumping the well more slowly until sand no longer appears or by abandoning the well.

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[1] See also BROWN, E. D., "Restoring Well Capacity with Chlorine," *Jour. A W. W.A.*, May, 1942, p. 698; and WHITE, H. L, Rejuvenating Wells with Chlorine," *Civil Eng.*, May, 1942, p. 263.

[2] See also FRIGATE, G. L., "Sand Removal from Well Water," *Water Works & Sewerage*, April, 1942, p. 161.

[3] See JESSUP, A. H., "Sand Removal from Well Water Based on Centrifugal Flow," *Water Works Eng*, Sept 11, 1910, p 1156

[4] BUNKER, G. C., "How to Determine Sand in Well Water," *Water Works & Sewerage*, June, 1943, p. 228.

CHAPTER VI

RAINFALL AND RUNOFF

119. Hydrology. Hydrology covers the science of water, its properties, occurrence, phenomena, and distribution. It deals primarily with precipitation and its disposal on and under the surface of the earth.

When rain starts to fall upon the surface of the earth, that part which is not immediately evaporated is absorbed into or passes into the soil. When the rate of supply exceeds the capacity of the soil to absorb the moisture, it may collect in little puddles formed by surface irregularities and vegetable cover. As the supply exceeds the capacity of the puddles, they begin to overflow and runoff occurs into the natural or artificial drainage system of the area. The capacity of the soil in a specified condition, to absorb or take in moisture, is known as the "infiltration capacity." The effect of the puddles is defined as "surface detention."

The infiltration capacity of a given soil is affected by the physical characteristics, the amount of soil moisture present at a given time, the prevailing plant-root structure present, and other factors. In general, the infiltration capacity becomes less and less as the rainfall continues.

The concepts provided by this approach to a determination of runoff resulting from a given storm are gradually removing the science of hydrology from an era of empiricism to an era of factual analyses.

120. Source of Water Supply. Rain, snow, hail, and sleet are precipitated upon the surface of the earth as meteorological water and may be considered as the original source of all water supplies. Part of the water becomes available for direct use in stream flow and ground water, and part is lost through processes of evaporation, transpiration, and deep seepage. The evaluation of the amount of water that may be available for the use of concentrated populations and the quantity of water that will produce damaging effects through erosion or flood are important functions of water-supply engineering. It is axiomatic that the amount of water available for direct use will be the difference between the total precipitation and the losses. However, exact determinations involve the effect of so many indeterminate factors that even the most experienced engineers encounter difficulties in arriving at reliable conclusions. It is important to emphasize the fact that there is no easy solution for the determination of the amount of water available and that, until an engineer becomes thoroughly competent, formulas and charts may be used only as a guide and experience and knowledge of this interrelation of the various factors must be acquired.

121. Precipitation. Precipitation occurs when moisture is transferred from the atmosphere to the earth. The capacity of the atmosphere to retain water vapor is a function of the temperature; therefore, when the temperature of the atmosphere falls below the dew point, the temperature at which the atmosphere becomes saturated with moisture, precipitation results. The factors involved in the phenomenon are many, and their interrelation is complex. Some of the factors are temperature, barometric pressure, prevailing winds, mountain barriers, relative position of land and water areas, altitude, and cyclonic storms.

122. Measurement and Records. The measurement and recording of precipitation is chiefly a governmental function; although many private companies and institutions keep such records for their own information. In the United States the work is done officially by the Weather Bureau of the Department of Agriculture. The data are published as follows: a *daily weather map* for each of several principal stations; *Climatological Data*, published monthly for 45 climatological sections and yearly in the form of a summary; the *Monthly Weather Review;* and an *Annual Report of the Bureau.* The Canadian Meteorological Service publishes daily weather maps, the *Monthly Weather Review*, and an *Annual Report.* Measurement of precipitation is made with standard instruments of the manual or automatic-recording type. A description of the instruments together with instructions for placement and manipulation may be found in *Circular E*, Instrument Division, U.S. Weather Bureau.

123. Amount of Rainfall. Topographic and climatic conditions in the United States are widely variant, and there are large areas of land remote from oceans or lakes. These factors together with others cause a large variation in the amount and distribution of rainfall. The mean annual precipitation, as shown in Fig. 57, varies from about 2 in. in sections of the Southwest to 100 in. or more in the extreme Northwest. Precipitation records at a few long-time Weather Bureau stations are given in Table 34.

The mean annual rainfall may be used as an index of the probable availability of water supply in a region. The actual rainfall in any one year may, however, deviate widely from the mean. The annual rainfall for each of a number of consecutive years may be continuously above or below the average. A long-continued period below the average may prove disastrous to a water supply. Many failures occurred during the drought period of 1930 to 1934. Figure 58 shows a 10-year moving average of annual precipitation at Saint Paul, Minn., and illustrates the

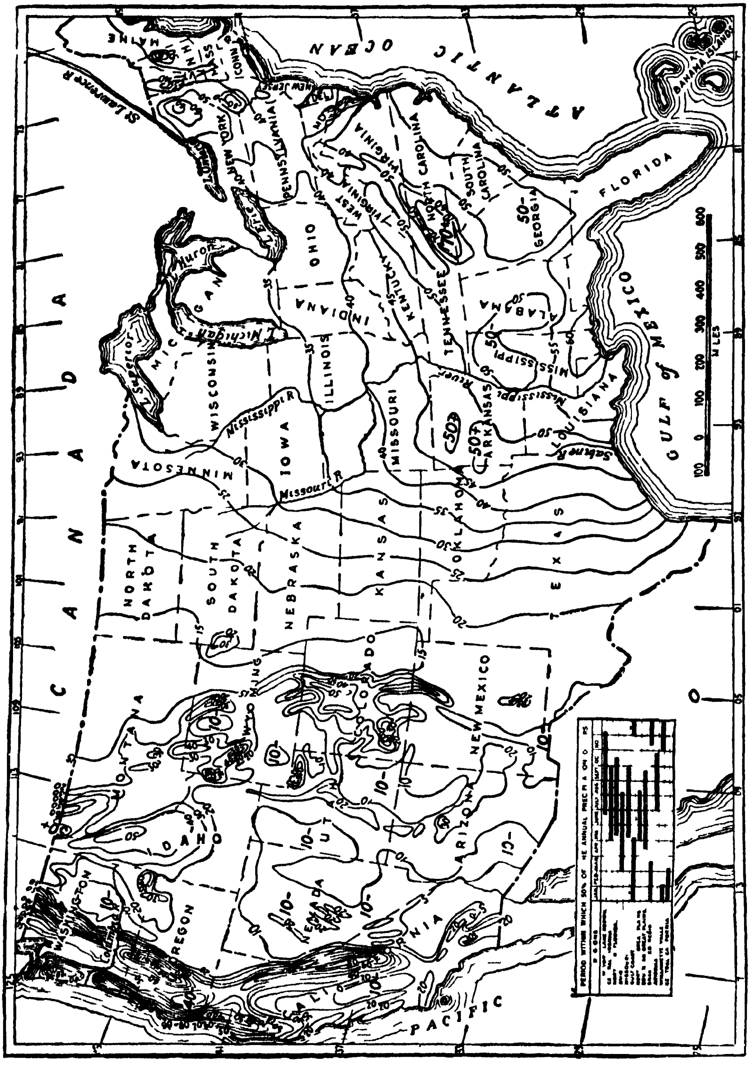

FIG 57 —Isohyetal map of United States

TABLE 34.—PRECIPITATION AT LONG-TIME WEATHER BUREAU STATIONS

Station	Period of record	Average annual, in.	Maximum		Minimum	
			Year	Per cent of average	Year	Per cent of average
Mobile, Ala..........	1871-1945	61.2	1881	151	1938	60
Yuma, Ariz.........	1870-1945	3.5	1905	326	1938	8.3
Los Angeles, Calif.....	1878-1945	15.7	1884	257	1898	31
Denver, Colo........	1872-1945	14.0	1909	164	1939	54
Peoria, Ill	1856-1945	35.0	1858	152	1910	66
Indianapolis, Ind....	1867-1945	39.6	1876	146	1934	63
Muscatine, Iowa...	1846-1945	38.9	1851	192	1901	56
Boston, Mass......	1818-1945	40.0	1863	170	1822	68
St. Paul, Minn......	1837-1945	27.3	1849	182	1910	37
St. Louis, Mo......	1837-1945	37.3	1858	184	1930	62
New York, N.Y. .	1826-1945	43.0	1859	139	1835	67
Portland, Ore.....	1872-1945	42.5	1882	159	1929	62
Yankton, S.D.....	1874-1945	25.0	1881	161	1894	58
Austin, Tex..	1856-1945	34.4	1919	188	1917	46
Milwaukee, Wis.. .	1844-1945	30.1	1876	167	1901	62

successive periods of high and low precipitation or the cyclical character of rainfall. Variations in seasonal rainfalls in a region, before recorded history, may be estimated by the development of a relation between the width of tree rings and the rainfall.[1] These synthetic records show the same general tendency toward successive high and low periods.

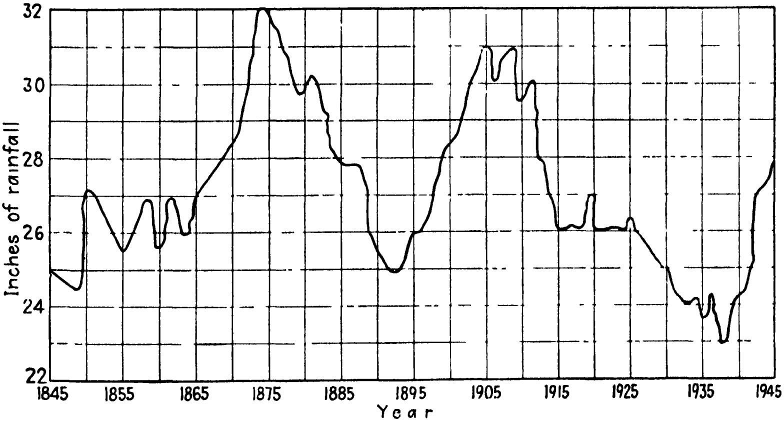

FIG. 58.—Ten-year moving average of annual precipitation at St. Paul, Minn.

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[1] DOUGLASS, A. E., *Sci. Monthly*, Vol. 37, p. 481, 1933.

U.S. Geological Survey, Water Supply Paper 772, 1936, gives an interesting analysis of studies to determine possible climatic changes in the United States. For the country as a whole there appears to be relatively small change in precipitation, but certain regions seem to have been affected in the last two or three decades by upward trends and others by downward trends. Average annual deviations from the mean are shown in Table 35.

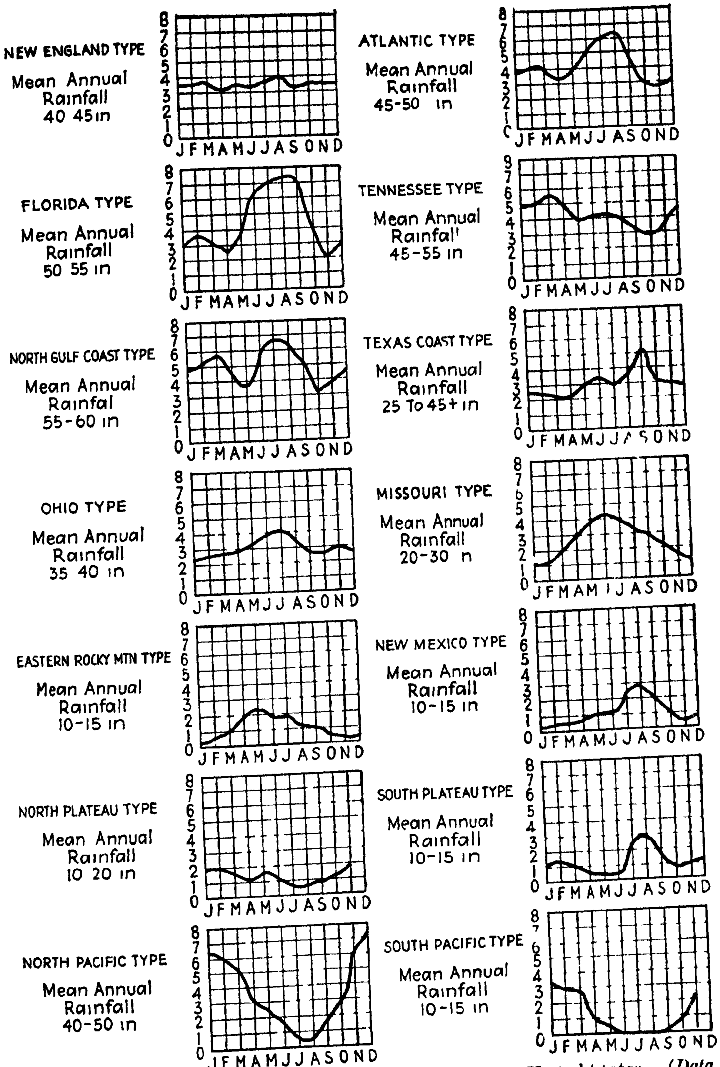
TABLE 35.—MEAN ANNUAL RAINFALL AND AVERAGE DEVIATION FROM MEAN

Region	Mean annual rainfall, inches	Average deviation, per cent
New England	40 to 45	11 to 14
Atlantic Coast	45 to 50	11 to 17
Florida	50 to 55	14 to 20
Tennessee River	45 to 55	11 to 14
North Gulf Coast	55 to 60	14 to 17
Texas Coast	25 to 45	17 to 23
Ohio River	35 to 40	11 to 20
Missouri River	20 to 30	14 to 30
Eastern Rocky Mountain	10 to 15	17 to 26
New Mexico	10 to 15	17 to 41
North Plateau	10 to 20	17 to 26
South Plateau	10 to 15	17 to 50
North Pacific Coast	40 to 50	11 to 26
South Pacific Coast	10 to 15	23 to 32

124. Distribution of Rainfall. The distribution of the annual rainfall of a region seems to be determined by climatic conditions peculiar to location, somewhat as shown in Fig. 59. These patterns illustrate in a general way the variations in the amount and occurrence of rainfall in different regions. There is often a relation between rainfall distribution and seasonal demand for water supplies, particularly in cases where storage problems are involved and where domestic supplies are used for irrigation purposes.

Hydrological considerations make it convenient to divide the year into three seasons or periods: (1) replenishing, September through November; (2) storage, December through April; and (3) growing, May through August. Figure 59 shows the distribution of rainfall from different parts of the United States by monthly variations.

125. Rate and Frequency of Intense Rainfall. The water-supply engineer is concerned with the rate at which rain falls in a single storm or series of storms and also with the frequency with which excessive precipitations occur. Since he is charged with the safety and economical



NEW ENGLAND TYPE
Mean Annual Rainfall
40 45in

FLORIDA TYPE
Mean Annual Rainfall
50 55 in

NORTH GULF COAST TYPE
Mean Annual Rainfal
55-60 in

OHIO TYPE
Mean Annual Rainfall
35 40 in

EASTERN ROCKY MTN TYPE
Mean Annual Rainfall
10-15 in

NORTH PLATEAU TYPE
Mean Annual Rainfall
10 20 in

NORTH PACIFIC TYPE
Mean Annual Rainfall
40-50 in

ATLANTIC TYPE
Mean Annual Rainfall
45-50 in

TENNESSEE TYPE
Mean Annual Rainfal'
45-55 in

TEXAS COAST TYPE
Mean Annual Rainfall
25 To 45+ in

MISSOURI TYPE
Mean Annual Rainfall
20-30 n

NEW MEXICO TYPE
Mean Annual Rainfall
10-15 in

SOUTH PLATEAU TYPE
Mean Annual Rainfall
10-15 in

SOUTH PACIFIC TYPE
Mean Annual Rainfall
10-15 in

Fig. 59 Typical distribution of monthly precipitation in the United States (*Data from Part III of Report of Water Planning Committee National Resources Committee.*)

design of structures that he builds for the control of water, he must provide against damage or failure from floods or excessive erosion. Many flood-flow formulas have been derived for various sections of the world. Some of these apply only to metropolitan areas for use in connection with the design of airports and storm sewers where the water from one storm is usually cleared away before the occurrence of a second storm or a maximum flow is reached, usually in less than 2 hr. after the beginning of a storm.

Within proper limitations the intensity of rainfall in inches per hour (i) for the type of watershed usually encountered in water-supply studies may be expressed as

$$i = \frac{C_1}{t^n} \tag{1}$$

where C_1 = a coefficient
n = an exponent whose values depend upon locality
t = duration of the rainfall, min.

Frequency of occurrence may be included in the formula by changing it to the expression

$$i = \frac{KT^x}{t^n} \tag{2}$$

where K = a coefficient the value of which depends on locality
T^x = a frequency factor indicating average frequencies of once in T years

The value of the exponent x depends upon locality.[1]

Table 36 gives illustrative values of the coefficients and exponents that have been determined by Merrill Bernard from rainfall studies in several localities.[2] The Bernard formula is empirical and therefore must be used wisely for determinations of actual rainfall-runoff relations.

126. Rate of Rainfall on a Drainage Area. The average rate of rainfall over a large drainage area is usually determined from observations at a number of widely separated stations. The reports from the stations are usually not in agreement. Thiessen[3] has devised an accepted method for averaging such observations into a general average applicable to the entire drainage area. The method is best explained by the solution of an example.

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[1] BERNARD, M. M., "Formulas for Rainfall Intensity," *Trans. A.S.C.E.*, Vol. 96, p. 592, 1932. Charts for K and n are shown in this reference.

[2] *Ibid.*

[3] THIESSEN, A. H., "The Precipitation Averages for Large Areas," *Monthly Weather Rev.*, July, 1911, p. 1082. See also HORTON, R. E., "Determining the Mean Precipitation on a Drainage Area," *Jour. New Engl. Water Works Assoc.*, Vol. 38, p. 1, March, 1924; BELKNAP, J. B., "A Comparison of Methods for Determining the Areal Mean Precipitation on Drainage Areas," *Jour. New Engl. Water Works Assoc.*, Vol. 46, p. 272, 1932.

TABLE 36.—ILLUSTRATIVE VALUES OF COEFFICIENTS AND EXPONENTS IN THE FORMULA $i = KT^x/t^n$ EAST OF 103D MERIDIAN

Region	K		x	n	
	t = 5 to 60 min.	t = 60 to 1,440 min.	t = 5 to 1,440 min.	t = 5 to 60 min.	t = 60 to 1,440 min.
New England..........	5.5 to 8.5	15 to 35	0 19 to 0.23	0.44 to 0.45	0.70 to 0.80
North Atlantic.........	8.5 to 9.0	35 to 45	0.17 to 0.23	0.43 to 0.44	0.79 to 0.82
Middle Atlantic........	9.0 to 9.5	45	0.19 to 0.21	0.41 to 0.42	0.79 to 0.82
Southeast..............	10.0 to 11.0	40 to 60	0.16 to 0.19	0.38 to 0.42	0.78 to 0.82
Ohio Basin.............	8.0 to 9.0	45 to 60	0.16 to 0.19	0.41 to 0.42	0.78 to 0.88
Upper Mississippi.......	8.5 to 9.5	40 to 45	0.17 to 0.20	0.43 to 0.47	0.78 to 0.85
Lower Mississippi.......	9.0 to 11.0	40 to 50	0.15 to 0.17	0.39 to 0.42	0.70 to 0.80
Missouri River.........	7.5 to 10.0	45 to 60	0.19 to 0.20	0.44 to 0.47	0.78 to 0.90
Southwest Mississippi...	7.5 to 9.5	40 to 60	0.18 to 0.22	0.41 to 0.44	0.75 to 0.87
Western Gulf...........	7.0 to 11.0	40 to 65	0.17 to 0.23	0.38 to 0.40	0.70 to 0.83

Let Fig. 60 represent a drainage area with rainfall observation stations at *A*, *B*, *C*, *D*, and *E*. Lines are drawn joining the points of observation, and perpendicular lines are erected that bisect the bases of the triangles thus formed. The areas that are to be allotted to each multiplied by the rate of precipitation observed at its corresponding station and divided by the total area of the watershed will give the approximate average rate of rainfall over the watershed.

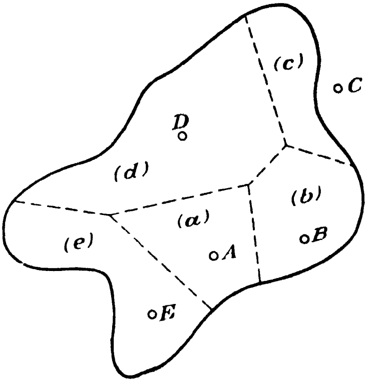

FIG. 60.—Diagram of drainage area with rainfall observation stations at *A*, *B*, *C*, *D*, and *E*.

127. Disposal of Precipitation. The precipitation that reaches the surface of the earth is disposed of in two ways, *viz.*, stream flow and losses. The stream-flow portion divides into surface flow and percolation. Runoff derived from ground water is sometimes called "base flow" or "sustained flow." The characteristics of the flow in a stream are greatly affected by the relative amounts of water derived from surface flow and ground-water flow. If the greater part is surface runoff, the flow will tend to be regular and well sustained even through drought periods.[1] Table 37 lists the physical factors affecting precipitation and its disposal.

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[1] The separation of stream flow into surface- and ground-water components is frequently of considerable importance in hydrological studies of water-supply problems. A discussion and bibliography is contained in *U.S. Geol. Survey, Water Supply Paper* 772, pp. 111*ff.*, 1936.

The losses may be classed as evaporative losses and deep seepage. The evaporative losses include direct evaporation from water and land surfaces, transpiration and, interception. Deep seepage comprises the rainfall that is lost to a given drainage area through seepage to a deep underground reservoir or by escape, through subterranean channels, to another drainage area. Rainfall that infiltrates into the soil and later

TABLE 37.—FACTORS AFFECTING STREAM FLOW[1]

Factor	Remarks
Precipitation	The amount of rain or snow, its distribution throughout the year; the intensity and manner of its occurrence; the character, extent, direction, and duration of storms
Temperature	Variations. The relation of temperature variations to the occurrence of precipitation. The accumulation of snow and ice due to low temperature. The existence of frozen ground at the time of heavy rains
Topography	The slope and the character of the area, whether rough or smooth
Geology	Relative imperviousness and the depth, slope, and thickness of surface and substrata. The character of the outlet of pervious strata whether emptying into this or other watersheds. The relative imperviousness of the channel of the stream. The presence of sand and gravel in the bottom of the channel
Ground surface	Extent, character, and state of cultivation of vegetation
Natural storage	Lakes, ponds, swamps, marshes. Porosity of surface and substrata and ability to store water
Watershed	Its size and shape. Prevailing winds. Direction relative to path of storms. Relation to mountains. Proximity to ocean or large lake. Glaciers or snow-capped mountains
Character of stream and tributaries	Slope; falls and rapids; pondage; regimen of channel. Arrangement of tributaries—joining the main stream at various points along its course or concentrated in fanlike arrangement at a common point of discharge
Artificial control of the stream	Dams and storage reservoirs. Dikes, levees, piers, abutments, and other encroachments
Wind	Intensity, direction, and modification by mountains and forests
Ice evaporation	Winter conditions. Gorges and floods. Temperature, humidity, wind, and other factors

[1] Suggested by summary in *Univ. Wis.*, *Bull.* 425, by D. W. Mead.

appears as base flow in the surface-drainage system of the watershed on which it falls is not considered a loss.

The quantitative evaluation of the amounts of evaporative losses is extremely complex. The rate at which evaporative losses occurs is dependent upon many factors. It may, and usually does, vary within very short intervals of time as the controlling factors change.

128. Evaporation from Water Surface. Direct evaporation from water surface is influenced by temperature, barometric pressure, mean wind velocity, vapor pressure of saturated vapor, and vapor pressure of saturated air. Direct evaporation from land surfaces is affected by the same factors plus effects due to the character of the surface, the culture thereon, the topography, and others.

Several formulas have been developed by experimenters in different parts of the country for evaporation from water surfaces. One of the most useful for general application is that proposed by Rohwer.[1] In a slightly modified form the formula is

$$E = C(1.465 - 0.0186B)(0.44 + 0.118W)(e_s - e_d) \qquad (3)$$

where E = evaporation, in. per 24 hr.
C = a coefficient
B = mean barometer reading, in. of mercury at 32°F.
W = mean velocity of ground wind or water-surface wind, m.p.h.
e_s = mean vapor pressure of saturated vapor at temperature of water surface, in. of mercury
e_d = mean vapor pressure of saturated air at dew point, in. of mercury (see Table 38 and note)

The formula was derived from observations of the evaporation from pans. The coefficient C is used to correct the formula for reservoir evaporation. An average value of C is about 0.75. Valuable digests of available evaporation data are contained in a symposium on "Evaporation from Water Surfaces" by Carl Rohwer and Robert Follansbee[2] and "Evaporation from Lakes and Reservoirs" by Adolph Meyer.[3] Representative values of the amount of evaporation selected from this symposium are given in Table 39.

Example: Find the daily evaporation from a reservoir water surface when the following conditions prevail: elevation above sea level, 475 ft. (B = 29.51 in.); air temperature, 68°; water temperature, 72°; wind velocity, 2 m.p.h.; relative humidity, 0.68.

Solution:

$$\begin{aligned} E &= 0.75\,(1.465 - 0.0186 \times 29.51)(0.44 + 0.118 \times 2)(0.785 - 0.68 \times 0.685) \\ &= 0.75 \times 0.916 \times 0.676 \times 0.319 = 0.148 \text{ in. per day} \end{aligned}$$

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[1] *U.S. Dept. Agr. Tech. Bull.* 271, 1931.

[2] *Trans. A.S.C.E.*, Vol. 99, p. 673, 1934.

[3] Published by the Minnesota Resources Commission.

129. Evaporation from Land Surface. The rate of evaporation from a land surface is affected by the same factors as evaporation from a water surface; in addition it is affected by the character of the surface, the culture thereon, its topography, the amount of water in or on the watershed, soil moisture, and other factors. The difficulties of expressing rates of evaporation from land surfaces with precision are, therefore, great. Various investigators have studied and reported their observations.

TABLE 38.—PRESSURE OF SATURATED WATER VAPOR AT DIFFERENT TEMPERATURES[1]

Air temperature, °F.	Vapor pressure, in. of mercury	Air temperature, °F.	Vapor pressure, in. of mercury
0	0.0383	50	0.360
5	0.0491	55	0.432
10	0.0631	60	0.517
15	0.0810	65	0.616
20	0.103	70	0.732
25	0.130	75	0.866
30	0.164	80	0.926
35	0.203	85	1.201
40	0.247	90	1.408
45	0.298	95	1.645
		100	1.916

[1] *U.S. Weather Bureau, Bull.* 235.

Interpolations in this table do not give exact values but are sufficiently accurate for ordinary calculations. Values of e_s in the Rohwer formula may be determined from this table. Values of e_d are determined by multiplying tabular values by the relative humidity.

Thornthwaite[1] proposes a formula as follows:

$$E = \frac{17.1(e_1 - e_2)(u_2 - u_1)}{T + 459.4}$$

where E = evaporation, in. per hr.
e_1 and e_2 = vapor pressures, in. of mercury at the lower and upper levels, respectively
u_1 and u_2 = wind velocities, m.p.h. at lower and upper levels, respectively
T = temperature, °F.

A summary of some of the results of studies of evaporation from various land surfaces is presented in Table 40. The students and the engineer must base their conclusions for design on data similar to those presented in this table. A diagram prepared by Meyer for the purpose of indicating the relation between evaporation from land surfaces and other factors is shown in Fig. 61.

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[1] *Monthly Weather Rev.*, January, 1939, p. 6.

TABLE 39.—RESERVOIR SURFACE EVAPORATION[1]

Station	Elevation, feet	Years	Temperature of air, degrees Fahrenheit	Wind velocity			Relative humidity, per cent	Reservoir surface evaporation, inches			Percentage range of annual evaporation	Coefficient for pan[2]
				Nearby U.S. Weather Bureau Station		At the pan, miles per hour		April to September	Annual	Maximum per month		
				Height, feet	Velocity, miles per hour							
United States:												
Ithaca, N.Y.	800	1918 to 1930	47	100	9.1	1.8	78	17.11	22.54	4.17	90 to 114	0.69
Washington, D.C.	280	1915 to 1917	54	85	6.5	2.3	69	23.52	34.53	4.87	97 to 103	0.69
Chapel Hill, N.C.	500	1921 to 1930	61	...	....	1.1	69	20.03	28.56	4.71	93 to 109	0.69
Madison, Wis.	860	1906 to 1911	46	78	10.0	...	75	12.91	19.82	3.04	97 to 102	0.83
Columbus, Ohio	763	1918 to 1930	52	230	10.7	2.0	74	19.21	26.81	4.94	89 to 108	0.69
Columbus, Mo.	750	1916 to 1927	54	84	8.0	1.5	71	20.28	28.13	5.31	88 to 114	0.69
Grand Forks, N.D.	820	1905 to 1920	38	58	9.8	...	79	21.72	27.07	5.82	90 to 117	0.83
Rapid City, S.D.	3,240	1916 to 1921	46	...	....	2.0	58	25.61	36.43	7.02	92 to 112	0.69
Lincoln, Neb.	1,250	1917 to 1930	51	81	10.0	4.1	69	32.06	42.04	9.92	87 to 118	0.69
Mitchell, Neb.	4,080	1911 to 1929	47	...	....	7.0	..	34.17	41.78	9.28	83 to 118	0.44
Lawrence, Kan.	825	1916 to 1919	55	...	....	4.4	..	31.25	44.80	7.62	....	0.69
Manhattan, Kan.	1,010	1924 to 1929	54	...	....	3.4	..	30.14	42.65	8.06	91 to 112	0.69
Austin, Tex.	475	1916 to 1929	68	148	7.5	2.0	68	28.72	42.47	8.49	77 to 116	0.83
Amarillo, Tex.	3,680	1907 to 1919	56	49	13.0	9.6	56	49.00	66.00	12.63	84 to 109	0.94
Denver, Colo.	5,340	1916	49	...	7.8	4.5	64	38.00	52.15	8.20	....	0.99
Salt Lake, Utah	4,250	1928 to 1930	52	203	6.2	3.6	50	40.67	50.94	10.10	95 to 103	0.69
Yuma, Ariz.	127	1917 to 1929	69	54	5.2	1.3	44	36.29	53.45	9.55	91 to 116	0.69
Independence, Calif.	3,800	1909 to 1911	56	28	6.6	...	45	38.61	55.26	8.00	....	0.83
Salton Sea, Calif.	230	1910	73	...	....	4.1	40	66.35	97.10	13.10	....	0.59
Corvallis, Ore.	235	1922 to 1930	52	...	....	1.5	..	21.88	30.68	5.92	91 to 106	0.69
Maunawili, T.H.	250	1921 to 1930	72	...	....	1.2	..	25.74	42.58	5.68	93 to 112	0.69
San Juan, P.R.	82	1919 to 1930	78	...	11.8	...	78	31.34	55.29	6.54	89 to 107	0.66
Canada:												
Keewatin, Ont.	.....	1913 to 1928	36	175	6.0	...	72	11.24	14.29	3.84	82 to 117	0.83
Edmonton, Alta.	.....	1918 to 1922	..	...	....	....	..	18.65	24.00	5.46	....	0.84
Egypt:												
Alexandria	.....	1920 to 1929	68	...	5.2	...	72	.....	28.76	.....	....	0.50
Atbara	.....	1905 to 1929	83	...	7.6	...	29	.....	123.67	.....	....	0.50
Turkestan:												
Andidjan	1,500	1920 to 1929	..	...	....	...	..	29.83	37.99	.....	....	0.95

[1] Data by Robert Follansbee, see *Trans. A.S.C.E.*, Vol. 99, p. 708, 1934.

[2] Evaporation from widespread water surface = coefficient for pan × measured evaporation from pan.

130. Transpiration, Consumptive Use, and Interception. Transpiration and consumptive use have been defined by the Committee on Absorption and Transpiration of the American Geophysical Union as follows:[1]

Transpiration. The process by which water vapor leaves the living plant body and enters the atmosphere.

TABLE 40.—EVAPORATION FROM LAND SURFACES[1]

Character of Surface	Relative Rates of Evaporation[2]
Bare ground	1.00[3]
Grain fields	0.80
Grassland	0.70
Light forests	0.60
Dense forests	0.2 0.4

[1] MEYER, ADOLPH F., "Elements of Hydrology," p. 244, John Wiley & Sons, Inc., New York, 1928.

[2] The total annual evaporation depends not only on the rate of evaporation but also on the available moisture. Meyer (*ibid.*, p. 246) has concluded that the annual water losses from rainfall, including evaporation, interception, and transpiration, do not differ widely, irrespective of whether the ground surface is covered by cultivated grains, grass, brush, or forest growth.

[3] For saturated soil the rate of evaporation may be taken the same as for free water surface.

Consumptive Use. The quantity of water per annum used by either cropped or natural vegetation in transpiration or building of plant tissues together with water evaporated from adjacent soil, snow, or from intercepted precipitation.

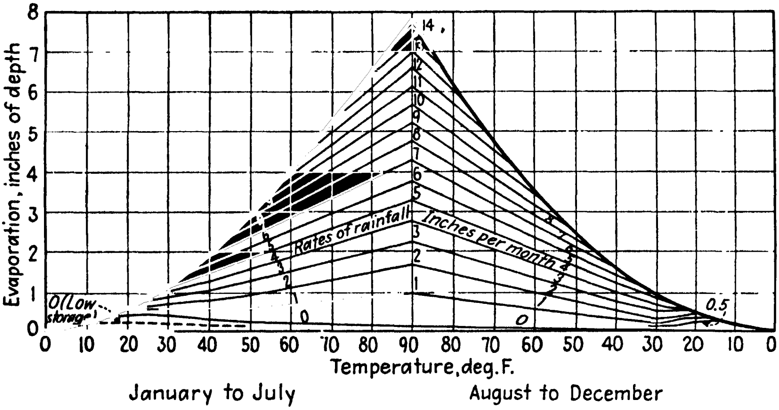

FIG. 61.—Evaporation from land areas for various temperatures and rates of rainfall. (*From A. F. Meyer, Trans. A.S.C.E. Vol.* 79, *p.* 1099, 1915.)

Interception is the amount of water per annum caught by vegetation (or structures) which is evaporated before reaching the ground.

The definition for consumptive use includes all the water losses from land areas and is of greater convenience to the water-supply engineer than the separate consideration of the three items of transpiration, soil evaporation, and interception that compose it. Many factors influence the amount of consumptive use; they include temperature, depth of water table, and kind of vegetation. A number of experiments have been and are being carried on to determine the amount of consumptive use. Certain cropped areas in the West show values of 1.2 to 3.0 acre-ft. per acre per year.[1]

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[1] *Trans. Am. Geophys. Union, Part* II, p. 286, 1935.

Taylor[2] states:

> In Southern California loss of water by transpiration and evaporation (consumptive use) from areas of native vegetation ranges from as low as 1 acre-ft. per acre per year or less to as high as 6½ acre-ft. per acre per year. The lower value applies to areas dependent entirely on rainfall; the higher value applies to moist areas with the water table at or close to the surface throughout the year.

Values as high as 9.63 acre-ft. per acre per year have been found for the consumptive use of tules and cattails. The use of water by vegetation is sometimes stated in terms of percentage of use compared with evaporation from open water. Table 41 gives a few such values. The values presented here are intended only to indicate a scale on the amount of loss by consumptive use. They should not be used without consultation of the literature on the subject.[3]

131. Infiltration. Since 1935, a great deal of scientific knowledge regarding infiltration has come to light. This has come about largely through the Research Division of the Soil Conservation Service and private investigators as Horton and Horner. As previously stated, no runoff occurs until the rainfall rate exceeds the rate at which a soil can absorb moisture. After a long dry period the infiltration capacity of a soil is at its maximum. This capacity is reduced during a single storm, and its recovery is rather slow between storms. Thus the effect of antecedent storms is important, especially when the time interval between storms is relatively short.

The phenomenon of infiltration is usually beneficial to water supplies for two principal reasons: (1) by reducing immediate runoff, thus tending to reduce flood flows, and (2) by providing underground storage, which tends to increase the base flow and hence the dry-weather flow.

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[1] "Consumptive Use of Water in Irrigation," *Trans. A.S.C.E.*, Vol. 94, p. 1399, 1930.

[2] TAYLOR, C. A., "Transportation and Evaporation from Areas of Native Vegetation," *Trans. Am. Geophys. Union*, 1934.

[3] Bibliographies may be found in *Trans. Am. Geophys. Union*, 1932 1946. See also MEYER, A. F., "Elements of Hydrology," John Wiley & Sons, Inc , New York, 1928; *Trans. A.S.C.E.*, Vol. 79, pp. 1094–1097, 1914; HORTON, R E., *Monthly Weather Rev.*, September, 1919.

An understanding of the infiltration phenomenon is very valuable in the study of the probable yield of a watershed for probable maximum flood flows and also for maximum, average, and minimum available water supply. With adequate knowledge of the fluctuations of infiltration capacity under varying storm conditions, an engineer may arrive at very close approximations of probable runoff for a particular storm or for a series of storms. Infiltration capacity is usually measured in terms of inches per hour. The details of the techniques to be employed are beyond the scope of this book, but the student is strongly advised to familiarize himself with the literature pertaining to infiltration. A brief bibliography is given in the footnote.[1]

132. Rainfall-runoff Relations. The principles of hydrology are applied more frequently in waterworks practice to the study of stream flow than for any other purpose. As previously stated, the runoff is the residual of rainfall after the deduction of losses. Many formulas have been devised in an attempt to establish a relation between rainfall and runoff. Such formulas should be used with great caution; they may give reasonable values for conditions for which they were derived, but they are almost certain to give erroneous results when applied indiscriminately.

The *rational method* is useful, in the hands of an experienced engineer, for estimating peak flows from a drainage area for which rainfall records, but no stream-flow records, are available. The formula used is

$$Q = CiA \tag{4}$$

where Q = discharge, c.f.s.

C = a coefficient whose value depends upon losses due to infiltration, evaporation, and transpiration, soil characteristics, and all other variations in contributions of rainfall to surface runoff in given area

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[1] MUSGRAVE, BROWNING, and FREE, "Relative Infiltration and Related Physical Characteristics of Certain Soils." *U.S. Dept. Agr. Tech. Bull.* 729, July, 1940.

SHERMAN, L. K., "Derivation of Infiltration Capacity (f) from Loss Rates (fav.) Macoupin Basin, Illinois." *Trans. Am. Geophys. Union, Part* II, 1940, p. 541.

SHERMAN, L. K., "Comparison of f-curves Derived by the Methods of Sharp and Holtan and of Sherman and Mayer," *Trans. Am. Geophys. Union*, 1943, p. 465.

ZINGG, A. W., "The Determination of Infiltration-rates on Small Agricultural Watersheds," *Trans. Am. Geophys. Union*, 1943, p. 475.

KOHLER, KARL O., JR., "Land-use and Vegetative Cover as Factors Influencing Runoff," *Trans. Am. Geophys. Union, Part* I, 1944, p. 40.

SHERMAN, L. K., "Infiltration and the Physics of Soil-moisture" *Trans. Am. Geophys. Union, Part* I, 1944, p. 57.

LENZ, ARNO T., "Checking Runoff by Use of Inflow and Outflow Volumes," *Trans. Am. Geophys. Union, Part* VI, 1944, p. 1011.

HORTON, ROBERT E., "Infiltration and Runoff during the Snow-melting Season, with Forest Cover," *Trans. Am. Geophys. Union, Part* I, 1945, p. 59.

HOBBS, HAROLD W., "Runoff Behavior of Small Agricultural Watersheds under Various Land-use Practices," *Trans. Am. Geophys. Union, No.* 1, 1946, p. 69.

i = intensity of average rainfall in inches per hour for a duration equal to concentration time of drainage area

A = area of drainage basin, acres

It is apparent that the dependability of the method hinges upon the accuracy of the data and assumptions from which C and i are determined. The intensity of rainfall for a given duration is discussed in Sec. 125. The duration of rainfall is affected by the time of concentration, which depends upon the velocity of flow in the stream and hence upon the slope, roughness, and shape of the watershed and water channels. Apparent velocity may be increased by 30 to 50 per cent in the main channel owing to the more rapid rate at which a flood wave may move so that the concentration time and, therefore, the duration may be reduced.

TABLE 41.—PROPORTION OF WATER RETURNED TO ATMOSPHERE AS COMPARED WITH EVAPORATION FROM OPEN-WATER SURFACE[1] TAKEN AS 100 PER CENT

Vegetation covering	Ratio of water returned, per cent	Vegetation covering	Ratio of water returned, per cent
Sod	192	Mixed crops.	144
Small grass.	173	Bare soil.	60
Forest.	151		

[1] Credited to Woolney, Kansas State Board of Agriculture; see *Trans. A.S.C.E.*, Vol. 99, p. 51, 1934.

The value of C represents the portion of the rainfall that finds its way into the surface stream. For example, if a rain having an average intensity of 2 in. per hr. falling on an area of 10 acres produces a runoff of 10 c.f.s., C would have a value of 0.5.

It may be desirable to consider frequency in connection with the studies of peak flow. If so, the following formula is sometimes used:

$$C = C_{\max}\left(\frac{T}{100}\right)^x \qquad (5)$$

where T is the average number of years between occurrences of runoff of a given magnitude and x is an exponent depending on locality (see Table 36).

Values of $C_{\max}$ may be greater than unity as a result of a warm rain falling on a snow-covered watershed. Values of 1.70 have been observed. In the United States east of the 100th meridian, values of $C_{\max}$ will seldom go below 0.50.

The best method of determining the relation between rainfall and runoff is by a "bookkeeping" process. This method consists of calculating runoff by subtracting the summation of net losses from the total rainfall supply. The formula is

$$R = P - L$$

where R = runoff
P = precipitation
L = losses[1]

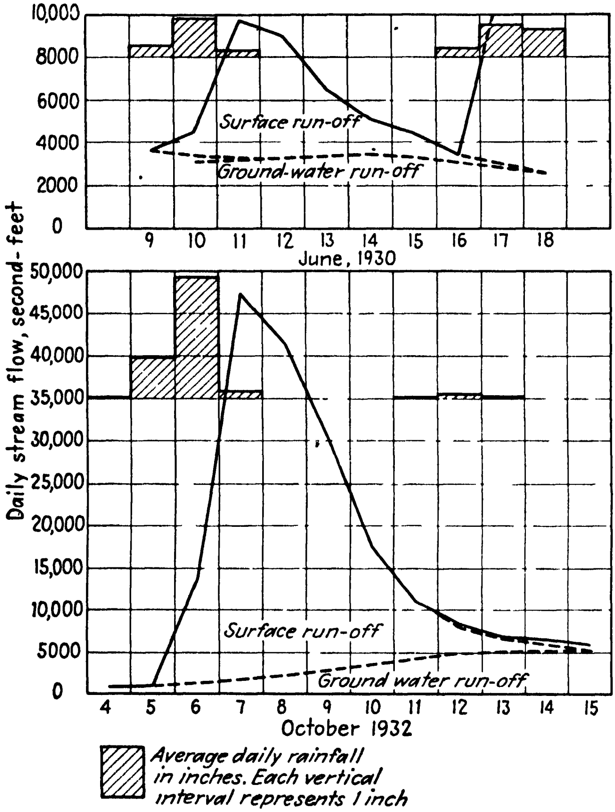

FIG. 62.— Unit hydrographs for Susquehanna River at Towanda, Pa. (*U.S. Geol. Survey, Water Supply Paper*, 772.)

133. Graphical Presentation of Runoff Data. Graphical methods useful in water-supply engineering for presenting runoff data include the hydrograph, the mass curve, and the unit hydrograph.

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[1] An example of this procedure is given by Hursh, Hoover, and Fletcher, "Studies in the Balanced Water Economy of Experimental Drainage Areas," *Trans. Am. Geophys. Union*, 1942, p. 509.

The hydrograph shows a chronological listing of stream flow. The abscissas are in units of time, and the ordinates either gage height or discharge. The part showing a rise in gage height or discharge is known as the "rising limb" and the falling part is known as the "recession limb." The highest point is called the "peak." Ordinarily the slope of the rising limb is steeper than the slope of the recession limb. The hydrograph may also show the base flow or that part of the stream flow derived from ground water. Hydrographs are shown in Figs. 62 and 93.

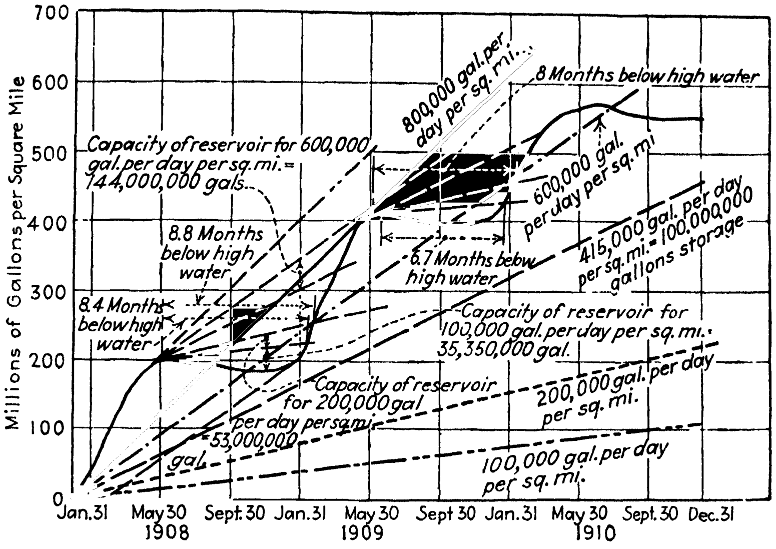

FIG. 63.—Mass curve of total runoff from Wachusett Reservoir Watershed, corrected for 20 per cent water surface. Note: All rates are in gallons per day per square mile.

The mass curve is the integral of a discharge hydrograph. The abscissas are in units of time and the ordinates, the volume of water which has passed the station during a given period. Mass curves are shown in Figs. 63 and 93.

The unit hydrograph, such as the one shown in Fig. 62, is a useful device that was first presented by L. K. Sherman in 1932. It is a curve that shows the rate of surface runoff resulting from rainfall within a unit of time. It is predicated upon the hypothesis that in a given basin the runoff occurring in a unit of time will produce hydrographs of similar pattern having equal bases and ordinates. In the hands of an expert, it is a most effective instrument, but a thorough understanding of its advantages and limitations is necessary and extremely important. A complete explanation of its use is beyond the scope of this book. The student is, therefore, referred to the selected bibliography[1] of the published data given herein. An example of a unit hydrograph is shown in Fig. 62.

134. Droughts. Droughts are defined[2] as periods during which precipitation is less than 85 per cent of the mean. They are ordinarily of greater importance to the water-supply engineer than the effects of unusually heavy storms. Available amounts of surface supplies and ground-water supplies may be seriously curtailed by drought. The effects of a drought, as defined, on available water supplies in an area having a low mean rainfall are apt to be much more serious than in an area having a high mean rainfall. Drought effects may continue long beyond the period of deficient rainfall as a result of the depletion of ground-water storage. The severity of a drought is increased greatly when high temperatures prevail over the affected areas. Unprecedented droughts prevailed over many sections of the United States from 1930 to 1934. The experiences of that period indicate what can happen and therefore have done much to emphasize the need for the careful analysis of the problems involved in predicting the minimum dependable supply.[3] Attempts have been made to demonstrate the existence of cyclical regularity of low and high precipitation, but the results in most cases are too vague to be dependable for forecasting purposes.

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[1] BERNARD, MERRILL, "An Approach to Determine Stream Flow," *Trans. A.S.C.E.*, 1935, p. 347.

Boston Society of Civil Engineers "Report of Committee on Floods," *Jour. Boston Soc. Civil Eng.*, September, 1930.

BRATER, E. F., "The Unit Hydrograph Principle Applied to Small Water-sheds," *Proc. A.S.C.E.*, September, 1939, p. 1191, discussion January, Report, 1940.

COLLINS, W. T., "Runoff Distribution Graphs from Precipitation Occurring in More Than One Time Unit," *Civil Eng.*, September, 1939, p. 559.

HORNER, W. W., and F. L. FLYNT, "Runoff from Small Urban Areas," *Trans. A.S.C.E.*, 1936, p. 140.

HOYT, W. G., and others, "Rainfall and Runoff Relation," *U.S. Geol. Survey, Water Supply Paper* 772.

McCARTHY, GERALD T., "The Unit Hydrograph and Flood Routing," U.S. Engineer Office, Providence, R. I., revised Mar. 21, 1939.

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SHERMAN, L. K., "Relation of Hydrographs of Runoff to Size and Character of Drainage Basins," *Trans. Am. Geophys. Union.*, 1932.

SHERMAN, L. K., "The Hydraulics of Surface Runoff," *Civil Eng.*, Mar. 10, 1940, p. 165.

SNYDER, FRANKLIN F., "Synthetic Unit Hydrographs," *Trans. Am. Geophys. Union*, Part I, pp. 447–454, 1938.

[2] HARROLD, L. L., "Frequency, Extent and Severity of Droughts," *Eng. News-Record*, Vol. 114, p. 978, 1935.

[3] See *U.S. Geol. Survey, Water Supply Paper* 772, 1936, pp. 255–268.

135. Runoff Records. Records of stream flow are available in certain federal and state publications, and often reliable private records may be obtained from power companies and water companies. The water-supply papers of the U.S. Geological Survey are the chief source of general information, although many states publish records serially or annually.

136. Flood-flow Estimates. The rational method may be used to estimate possible maximum flood flows. Other empirical formulas that do not involve the rainfall factor are also used. These formulas may be used to advantage in areas such as parts of the western United States where very limited basic hydrological data are available. The formulas are based on measured flows and are of the form

$$Q = CM^q$$

or discharge in cubic feet per second equals a coefficient C times a power q of the drainage area M, in square miles.

The modified Myers formula[1]

$$Q = 10{,}000p\sqrt{M}$$

where p is a variable percentage coefficient expressing the ratio of the maximum flood for a given stream to an assumed maximum for all streams. The discharge q in second feet per square mile is given as

$$q = \frac{10{,}000p}{\sqrt{M}}$$

The percentage p that applies to any particular region is termed the "Myer rating" for the area. The maximum Myer rating for the eastern part of the United States is rarely more than 60 per cent. Ratings for more than 900 streams are given by Jarvis.[2]

The unprecedented floods of 1936 in New England and of 1937 in the Ohio Valley have added to the experience of engineers and have emphasized the caution that must be exercised in the design of structures.

A discussion of methods for estimating flood flows and also data on maximum floods may be found in *U.S. Geological Survey, Water Supply Paper* 771, 1936. Methods[3] involving probability of magnitude and occurrence of rainfall and flood flows derived from statistics of records are sometimes used. *Water Supply Paper* 771 also contains several discussions on the value of these methods.

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[1] JARVIS, C. S., "Flood Flow Characteristics," *Trans. A.S.C.E.*, Vol. 89, p. 994, 1926.

[2] *Ibid.*, pp. 1003*ff.*

[3] HAZEN, ALLEN, "Flood Flows," John Wiley & Sons, Inc., New York, 1930; FOSTER, H. A., "Theoretical Frequency Curves," *Trans. A.S.C.E.*, Vol. 88, p. 142, 1924; GOODRICH, R. D., "Straight Line Plotting of Skew Frequency Data," *Trans. A.S.C.E.*, Vol. 91, p. 1, 1927; PETTIS, C. R., "Long Range Flood Predictions," *Eng. News-Record*, Vol. 116, p. 871, 1936; FULLER, W. E., "Flood Flows," *Trans. A.S.C.E.*, Vol. 77, p. 564, 1914.

CHAPTER VII

DAMS

137. Purposes. Dams[1] may be used in waterworks to form impounding, storage, or distributing reservoirs or for the control of stream flow. Control dams, sometimes called "diversion dams," are used to raise the water level of a stream enough to provide gravity transportation of water to pumps or reservoirs, to maintain adequate submergence over intakes, or to provide a suitable depth of water in the suction well of a pumping station.

It is necessary to obtain a permit from the federal government for the construction of a dam across a navigable stream and, in view of the New River Decision[2] by the United States Supreme Court, it is apparent that this stipulation includes any tributary to a navigable stream. In some states it may be necessary to obtain both a federal and a state permit.

The safety of the dam is usually of vital importance to the waterworks that depends on its functions for the source of water.

138. Types. Dams may be classified according to the materials of construction, as earth, including rolled fill, hydraulic fill, and semihydraulic fill; rock fill; rock-filled timber crib; masonry, including stone, plain concrete, and reinforced concrete; timber; and steel. They may be classified also in accordance with the basis on which they resist external stresses, as solid gravity, including straight, curved, and arched dams; hollow gravity, including slab and buttress, multiple arch, and multiple dome; and arch, including constant angle and constant radius.

139. Dam Sites. Topographically a dam to create a reservoir should be located where it can be constructed high enough to store the water needed. Methods for determining this height are discussed in Sec. 198.

The site must be carefully studied geologically. Foundation explorations are made by test pits and by drilling methods, some of which are explained in Chap. V. The character of the material encountered is studied from the standpoint of its composition, structure, stratification, permeability, and bearing power. Guided by the results of subterranean explorations and by observation of surface conditions, a competent geologist may determine the presence of fault planes and the probability of movement upon them. He also will give valuable advice concerning the suitability of the foundation materials in the matter of the effect upon them of weathering, erosion, and immersion. *Intelligent* application of the principles of soil mechanics is indispensible to the proper design of many dams. Especial care in the selection and preparation of the foundation is of vital importance to the safety of a dam.

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[1] See also CREAGER, W. P., J. D. JUSTIN, and JULIAN HINDS, "Engineering for Dams," John Wiley & Sons, Inc., New York, 1944.

[2] See *Jour. A.W.WA.*, February, 1941, p. 315.

140. Porous Foundations. Dams are safely constructed on porous foundations, the weight of the structure and the downstream apron being sufficient to resist uplift indicated by the "flow net" and the imperviousness of the material of the foundation being sufficient to prevent "piping." The flow net is a diagram composed of lines of equal pressure in a porous medium. A typical flow net is illustrated in Fig. 64. The rate of flow along lines *A*, *B*, and *C* is fixed by the hydraulic gradient, or the distance between lines of equal pressure potential. This distance can be increased by the construction of cutoff walls or by increasing the length of the apron. It is generally better to increase the length of the upstream apron since the weight of the water over it aids in overbalancing uplift pressure. If the velocity of flow is sufficiently great to move underground material, piping is taking place. This is most likely to occur along the "line of creep," *i.e.*, the line of contact between the dam and the foundation, because the resistance to flow is least along this line. The breaking of contact between the dam and its foundation is called "roofing "

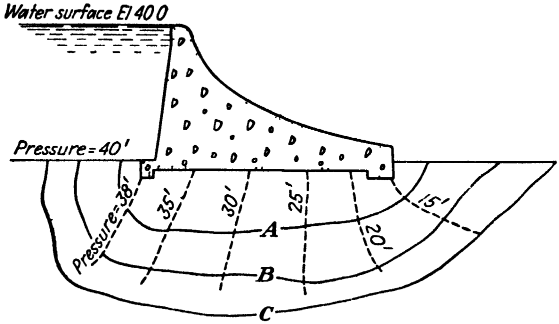

FIG. 64. Dam on porous foundation showing typical flow net.

EARTH DAMS

141. Earth Dams. Earth dams can be designed and constructed safely if proper attention is paid to important details. Some causes of failure of earth dams are summarized in Table 42. In a list of 35 failures given by Creager, Justin, and Hinds,[1] foundation failures account for 4, poor embankments for 8, inadequate spillways for 6, piping for 11, seepage along masonry or rock for 4, animals for 1, and broken outlet for 1. Each cause may be attributed to lack of attention to a detail in design, construction, or maintenance.

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[1] CREAGER, JUSTIN, and HINDS, *loc. cit.*

TABLE 42.—CAUSES OF FAILURES OF EARTH DAMS,[1] 1799–1931

Cause	Number of failures	Percentage of total
Inadequate spillway	52	33
Inadequate cutoffs	36	22
Improper compaction	8	5
Slopes too steep	6	4
Inadequate stream control during construction	5	3
Improper selection of materials	15	10
Earthquakes	3	2
Other miscellaneous and undetermined	34	21
	159	100

[1] HINDERLIDER, M. C., "Necessity for, and Penalties for Lack of, Supervision," *Trans. A.S.C.E.*, Vol. 98, p. 836, 1933.

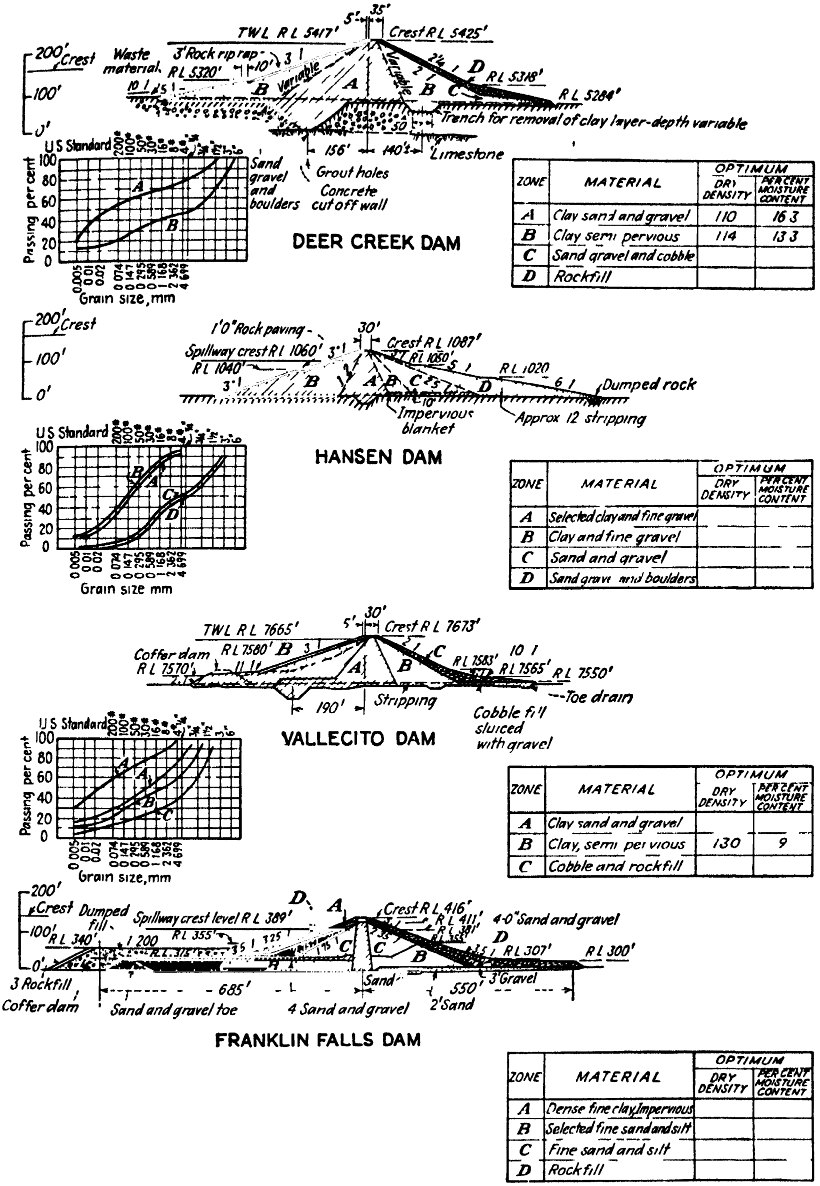
Earth dams may be classified in accordance with the methods of their construction as rolled fill, hydraulic fill, or semihydraulic fill. A rolled-fill dam is constructed by depositing the material in layers, 4 to 12 in. thick, and compacting each layer by rolling. The material for hydraulic-fill dams is transported from the borrow pit to the fill by means of a stream of water carried in a pipe or flume.

Sections through typical earth dams are shown in Fig. 65.[1] These sections illustrate some of the important details to be considered in the design of earth dams. They include (1) embankment slope, (2) slope protection, (3) top width, (4) freeboard, (5) berms, (6) quality and location of materials in the embankment, cutoff walls, and core wall, (7) drainage of downstream face and protection of upstream slope against sudden drop of water level in the reservoir, (8) prevention of piping, and (9) prevention of roofing.

Care in the construction of earth embankments as dams or levees to hold water must be emphasized. Careless or ignorant work, permitting leakage, may result in a slow cumulative erosion of the embankment terminating in failure. Earth dams must be carefully built.

142. Hydraulic-fill Dams. The embankment material for hydraulic-fill dams is sluiced or transported from the borrow pit to the fill by means

[1] See also KELSO, A. E., "Water Supply Practice," *Jour. A.W.W.A.*, July, 1942, p. 984.



DEER CREEK DAM

ZONE	MATERIAL	OPTIMUM	
		DRY DENSITY	PERCENT MOISTURE CONTENT
A	Clay sand and gravel	110	16 3
B	Clay semi pervious	114	13 3
C	Sand gravel and cobble		
D	Rockfill		

HANSEN DAM

ZONE	MATERIAL	OPTIMUM	
		DRY DENSITY	PERCENT MOISTURE CONTENT
A	Selected clay and fine gravel		
B	Clay and fine gravel		
C	Sand and gravel		
D	Sand gravel and boulders		

VALLECITO DAM

ZONE	MATERIAL	OPTIMUM	
		DRY DENSITY	PERCENT MOISTURE CONTENT
A	Clay sand and gravel		
B	Clay, semi pervious	130	9
C	Cobble and rockfill		

FRANKLIN FALLS DAM

ZONE	MATERIAL	OPTIMUM	
		DRY DENSITY	PERCENT MOISTURE CONTENT
A	Dense fine clay, Impervious		
B	Selected fine sand and silt		
C	Fine sand and silt		
D	Rockfill		

FIG 65 Profiles of American rolled-fill dams. (*From A. E. Kelso, Jour. A.W.W.A., July, 1942, p. 997*)

of a stream of water carried in a pipe or flume. A dike is maintained at the upstream and downstream faces, and the center is kept low, forming the central pool. The material is delivered from the sluices near the dikes and is allowed to flow toward the center. The heavier particles will be dropped first near the faces of the dam, and the finest will be carried into the central pool, thus procuring a gradation of materials. The finest materials form the core, or the impervious central portion, of the dam. Excess water from the central pool is carried away through controlled outlet pipes. Care is necessary to maintain the proper slopes from the outside toward the center in order to avoid high velocities or gullying, which might result in the coarse materials being carried into the center.

Slides occurring during construction or after completion, resulting from the high fluid pressures in the core due to the inability of the core to drain, are a source of danger in hydraulic-fill dams. Hazen[1] states that:

> To use the hydraulic method of dam construction successfully, it is necessary either to increase the toes until they are large and heavy enough to resist successfully the fluid pressure of the core or else to handle the core material so that the finest particles are wasted, leaving only the large ones, and in that way increasing the grain size to a point where drainage can be secured, while on the other hand remaining sufficiently impervious for practical purposes.

Semihydraulic-fill dams differ from hydraulic-fill dams in the manner by which the filling material is brought to the embankment. Means other than water are used to transport the materials to the fill. Water, however, is used to classify and finally place the material. In general, the same principles apply as for hydraulic-fill dams.

143. Embankment Slopes. Embankment slopes may be governed by one or by all three of the following considerations: (1) stability of the material under all probable conditions of moisture content, (2) bearing power of the foundation material, and (3) the resistance of the soil to the percolation of water.

It is unsafe to make generalized statements in connection with safe slopes for earth dams. The character and properties of materials and of foundations vary so widely that each case must be considered separately. Ordinarily, for structures 25 ft. or less in height, stable embankments may result if the upstream slope is built not steeper than 2½ horizontal to 1 vertical and the downstream slope, 2 horizontal to 1 vertical. For higher dams, investigations should be made to determine:[2]

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[1] HAZEN, ALLEN, "Hydraulic-fill Dams," *Trans. A.S.C.E.*, Vol. 83, p. 1713, 1919–1920.

[2] LEE, C. H., "Selection of Materials for Rolled-fill Earth Dams," *Proc. A.S.C.E.*, September, 1936, p. 1025. See also *Proc. A.S.C.E.*, December, 1936; January, February, March, and April, 1937.

1. Stability of soil against sloughing when saturated.
2. Watertightness or resistance to percolation.
3. Workability for construction operations.
4. Solubility of mineral constituents.
5. Bearing power of foundations.
6. Reasonable cost of handling, in excavation, transportation, spreading, and compacting.

Intelligent interpretation of laboratory tests recommended by engineers, expert in this field, should form the basis of final designs for major structures and questionable minor ones.[1]

144. Slope Protection. The slopes of an earth dam may require protection against erosion from wave action and surface runoff and against burrowing animals. The effect of wave action may be eliminated or reduced by the placement of stone riprap, concrete slabs or blocks, or willow mattresses on the upstream slope. The protection should extend from the top of the dam to 5 ft. below the lowest expected operating level. In important structures where riprap or concrete is used, a shoulder, or berm, should be provided at the base of the protection. Downstream slopes may be protected from erosion by loose rock fills or by planting with vines, shrubs, or grass. Plants that develop long root structures, which might penetrate too far into the dam, should be avoided. For long slopes, a berm wide enough to accomodate an intercepting gutter should be supplied. Loose rock fills, concrete, wood, or steel cores provide protection against burrowing animals. Fences may be necessary to keep grazing animals from destroying grass cover and making trails which become water channels during storms.

145. Top Width. The minimum top width W for a height H may be determined from the empirical formula $W = 2\sqrt{H} + 3$. If a roadway is to occupy the top of the dam, a width of at least 2 ft. greater than the traveled way should be provided on each side of the road.

146. Freeboard. Freeboard is the vertical distance from the water surface to the top of the embankment. This distance must be sufficient to prevent overtopping of the dam during high water, frost penetration, and damage due to wave action. The height required to prevent overtopping is fixed by the height of water allowed over the spillway. The height required for safety against frost will vary from nothing in temperate climates up to 8 ft. or more in cold climates. The freeboard necessary to prevent overtopping by waves should be measured from the highest water elevation anticipated. This freeboard may be computed from Stevenson's[2] formula $H = 1.5\sqrt{D} + (2.5 - \sqrt[4]{D})$ where H is the height of the waves, or the added freeboard required, and D is the horizontal length of water exposure or "fetch" of the wind, in miles.

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[1] "Laboratory Procedure in Testing Earth Dam Materials," *Bur. Reclamation, Tech. Mem.* 533, 1936.

[2] Stevenson, Thomas, "Design and Construction of Harbors," 2d ed., Edinburgh, 1874.

147. Berms. A berm is a horizontal break in the slope of the sides of an embankment, as shown in Fig. 65. Submerged berms are used on the upstream side of earth dams to support pavement and to increase stability of the structure. They may be used on the downstream side to interrupt the flow of rain down the face of the dam and also to increase the stability of the structure. When placed with a gentle slope parallel to the axis of the dam, the berm may serve as a roadway up the dam.

148. Classification of Materials.[1] Materials most readily available for the construction of earth dams and embankments are clay, sand, and gravel. Clay is practically impervious but is highly absorbtive. When saturated it tends to become fluid, it is slow to drain, and its cohesive and frictional resistances are greatly reduced. It shrinks in drying and is subject to cracking. Sand and gravel are pervious, drain easily, and their angle of repose, which is a measure of cohesive and frictional resistance, is not greatly affected by saturation. A mixture of gravel, sand, and clay, properly placed, forms a satisfactory material for an earth dam.

Reduction in volume of materials due to settlement, shrinkage, compaction, and consolidation is an important consideration in the preparation of designs and estimates. Compaction may reduce the volume of loose materials 25 or 30 per cent. An allowance of 5 per cent in height due to settlement is ample for dams in which ordinary care has been exercised in the placing of the materials.

In placing materials in the embankment the most impervious, except possibly the core wall, should be placed at the upstream face of the dam. The material of the dam may grow progressively more pervious from the upstream to the downstream face of the dam. In fact, it may be desirable to drain the downstream part of an earth dam by placing drainage pipes in the embankment, thus preventing the accumulation of water in it with resulting danger to the structure. The seepage line, *i.e.*, the level of the water in the dam, should be kept low on the downstream side of the core wall. The classification of materials in four earth dams is shown in Fig. 65.

149. Core Walls and Cutoff Walls. A core wall, or core, is a partition placed in a dam, usually near the center or toward the upstream face, to prevent or impede the passage of water or of burrowing animals through the dam. An improperly placed core wall may endanger the safety of the dam by holding water in the structure, thus materially decreasing its resistance to shear and sliding, and increasing the danger of the overturning of the core wall.

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[1] See also LEE, C. H., "Selection of Materials for Rolled-fill Earth Dams," *Trans. A.S.C.E.*, Vol. 103, p. 1, 1938.

Hollow, or cellular, cores have been used, as in the Sylvan Dam at Melbourne, Australia, to assure the dryness of the embankment on the downstream side of the core wall, to prevent overtopping of the embankment during construction, and for other reasons. A cellular core wall is constructed with hollow vertical chambers as close together as practicable in the downstream portion of the wall. The vertical chambers terminate at the bottom in a drainage tunnel which conducts the leakage away. All percolation through the downstream portion of the embankment is thus prevented. The hollow cylinders and the tunnel should be made large enough to permit inspection of the condition of the dam.

Materials used in core walls include concrete, steel, wood, and puddle or puddled earth. Since stresses on a core wall cannot be computed without making uncertain assumptions concerning movements of the dam, the determination of the core-wall thickness is a matter of judgment based on experience. Concrete core walls with a top thickness of 12 in. and sides battered 1 horizontal to 100 vertical have proved generally satisfactory. The core wall should be sufficiently high to extend above the frost line in the dam. Some cores extend above the top of the earth embankment to act as a parapet and factor of safety. During construction the earth pressures on the sides of the core can be balanced by maintaining the rising fill at the same elevation on each side of the core

Steel and wood are seldom used in core walls except for temporary installations since the alternate wetting and drying of the upper portion of the structure exposes the steel or wood to active corrosion.

Puddle is a definitely proportioned mixture of clay, sand, and water placed and tamped to form an almost impervious diaphragm. It may satisfactorily stop the flow of water, but it is not a deterrent to burrowing animals. Puddle cores, usually placed at or near the center of the dam, as indicated by material *A* in Fig. 65, are formed by placing a mixture of selected materials by a hydraulic process. Puddle cores have the advantage of flexibility and are less subject than concrete to rupture by unbalanced forces within the fill. Care is necessary in placing the core to avoid undesirable stratification of materials within it.

A cutoff wall is a low core or wall extending from a short distance in the dam above the porous stratum on which the dam is founded, down into the porous stratum to cut off flow through it. Such a wall is shown in Fig. 66.

150. Blankets. Blankets composed of a tough workable clay spread on the floor of the reservoir, under the fill or on the upstream slope, are often used to prevent or reduce seepage through or under an earth dam. For highly impervious material the thickness of the blanket should be at least 2 ft. Blankets are used when the expense of a core, by reason of excessive depth, is too great and when they can be used advantageously instead of a core with material unsuitable because of its perviousness. To determine the thickness of blankets that extend upstream from the dam, the following formula is sometimes used: $t = 2\text{ ft.} + 0.02d$, in which t is the thickness in feet and d is the distance in feet from the upstream end of the blanket to the dam. A cross section of a dam provided with a clay blanket is shown in Fig. 66.

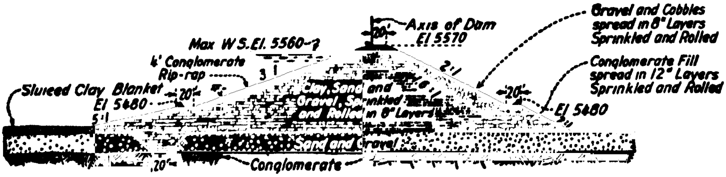
Gravel and Cobbles spread in 8" Layers Sprinkled and Rolled
Axis of Dam
20'
El 5570
Max W.S. El. 5560
4' Conglomerate Rip-rap
3:1
2:1
Conglomerate Fill spread in 12" Layers Sprinkled and Rolled
Sluiced Clay Blanket
20'
Clay, Sand and Gravel, Sprinkled and Rolled in 8" Layers
20'
El 5480
5:1
El 5480
2:1
Sand and Gravel
20'
Conglomerate

FIG. 66.—Method of providing imperviousness on porous foundations at Echo Dam. (*Courtesy of U.S. Bureau of Reclamation.*)

151. Safety of Earth Dams. Earth dams, however carefully constructed, have failed. Knowledge of the causes of failures may lead to their avoidance. A list of such causes is given in Table 42 and is discussed in Sec. 141.

MASONRY DAMS

152. Masonry Dams.[1] Masonry dams are classified according to type as:

Solid gravity: straight, curved, or arched.
Hollow gravity: slab and buttress, multiple-arch dome, and flat slab.
Arch: constant angle and constant radius.

Figure 67 shows four types of gravity dams designed for the American Falls site on the Snake River in Idaho. The gravity type was finally built. Dams may be further classified as storage and diversion and as overflow and nonoverflow.

The gravity dam receives its name from the fact that the weight of the structure plus the weight of water on it is sufficient to resist the forces acting to overturn, shear, or slide the structure. A straight gravity dam is a dam whose lines, in plan, particularly the axis of the dam, are straight. Curved, or arched, gravity dams are synonymous in that they are curved in plan, either upstream or downstream, but they resist overturning due to their weight and not as an arch. Various other types of gravity dams are illustrated in Fig. 67. Hollow gravity dams when compared with solid gravity dams[2] require less material, offer effective drainage, relieve uplift pressure, may slightly more than double the factor of safety against overturning, and the sliding factor is reduced not more than about 50 per cent The serious objections to them are the difficulties and cost of construction.

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[1] See also "Masonry Dams, a Symposium," *Trans. A.S.C.E.*, Vol. 106, p. 1113, 1941.

[2] See also LEWIS, J. W., "Hollow Dams," *Water and Water Eng.*, December, 1939, p. 569.

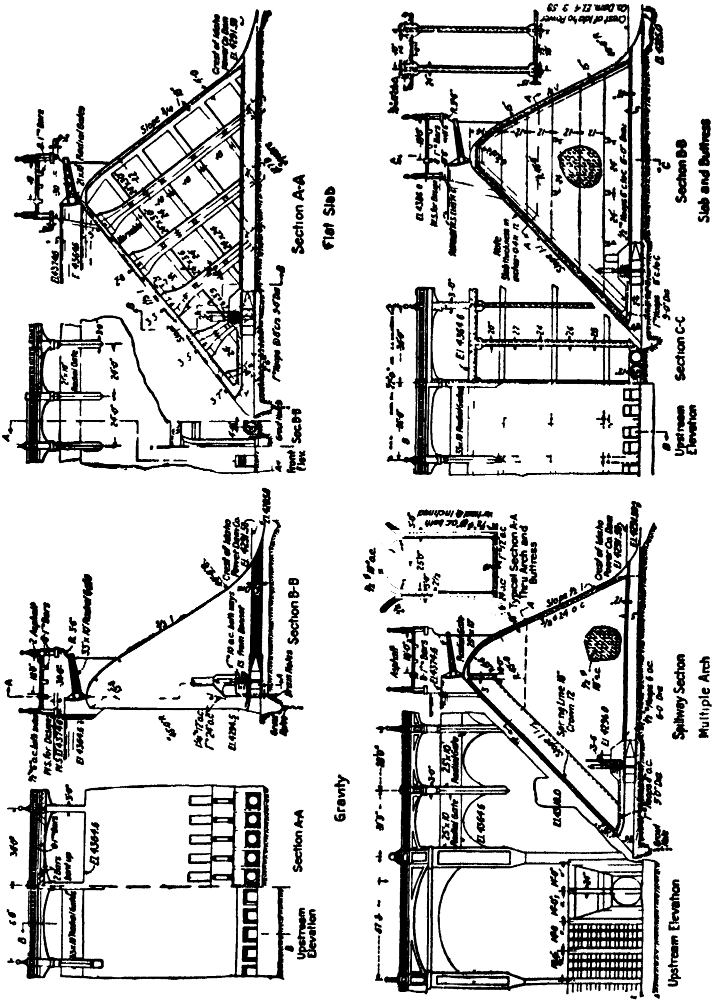

FIG 67 —Comparison of types of gravity dams

An arch dam is curved in plan with its ends resting on abutments in the hills on either side of the valley. The dam acts as an arch with its haunches resting on the abutments and the dam structure under compression In general, arch dams will have less material in them than a solid gravity dam. However, the cost of construction of the arch dam may be greater owing to form work and other difficulties.

153. Other Dams. A rock-fill dam[1] is an embankment of loose rock with a flow-retarding membrane somewhere in or on the embankment. Materials used for the membranes include concrete; earth, either paved or unprotected; timber; or steel.[2] There is a trend toward the placing of a layer of dry uncoursed rubble between the facing and the rock fill. The rubble furnishes a semirigid element subject to less settlement and provides good distribution of the water load to the rock fill. Settlement due to the consolidation of the fill and to the water load is an important factor in rock-fill dams.

The cross section of a rock-fill dam for average conditions should be about as follows: minimum top width, 15 ft.; upstream slope, 1.3 horizontal to 1 vertical; downstream slope, 1.3 to 1.4 horizontal to 1 vertical; thickness of rubble depends on conditions and good judgment. The cutoff wall provides a connection between the facing and the bottom and sides of the dam site. It should be built of dense concrete and extend to a depth sufficient to provide adequate resistance to percolation under the dam. The flow-retarding elements are extremely important to the safety of the structure. Facings must be designed to remain watertight while being adjusted to temperature changes and the effects of settlement which may extend over several years. Frequent inspections and repairs of facings should be anticipated. Central cores are subject to full water pressure and hence to considerable distortion and cracking from the effects of settlement. Being inaccessible, they cannot be repaired.

A rock-fill dam is useful at sites for which transportation of other materials is difficult, concrete is expensive, satisfactory earth materials are not available and where the site is favorable to the rock-fill type. If not properly designed and built, the frequent unwatering for repairs makes rock-fill dams undesirable for domestic water-supply purposes.

Wood and steel dams have little application to waterworks construction. They are generally short-lived and are suitable only for low dams.

FORCES ON DAMS

154. Forces Acting on Dams. Forces that must be resisted by dams include (1) water pressure, (2) weight of dam and superimposed water, resulting in foundation reaction, (3) flotation, (4) uplift, (5) erosion, (6) ice pressure, (7) earthquake shocks, (8) wind pressure, (9) wave pressure, and (10) subatmospheric pressures.

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[1] See also GALLOWAY, J. D., "The Design of Rock-fill Dams," *Trans. A.S.C.E.*, Vol. 104, p. 1, 1939.

[2] LOGEMAN, R. T., "Steel Facing for Dams," *Civil Eng.*, January, 1939, p. 7.

155. Water Pressure. The horizontal component of the water pressure can be computed from the expression $P = wAh_0$, where P is the total horizontal pressure, w is the unit weight of water, A is the area of the horizontal projection of the face of the dam on to a vertical plane, and h_0 is the vertical distance from the water surface to the center of gravity of the area A. The vertical component of the water pressure is equal to the weight of water vertically above any portion of the dam that is submerged. The direction and point of application of water pressure is normal to the face of the dam at the center of pressure.

156. Flotation and Uplift. Flotation, uplift, and foundation reaction are forces with vertical components acting on the dam. The magnitude of flotation is equal to the volume of water displaced by any portion of the dam submerged in and completely surrounded by water multiplied by the unit weight of water. Flotation will not come into play unless there is complete submergence of a part of the dam. Uplift is caused by the pressure of water confined within the interstices of the dam and connected by open channels, however small, to the water within the reservoir. The magnitude of the uplift can be expressed as $U = \Sigma wAh$, where w is the unit weight of water, A is the area of the vertical projection on a horizontal plane of all surfaces subject to uplift, and h is the head, or pressure of water on such surfaces. It is measured as the distance from the point of pressure to the hydraulic grade line of the channel in which the water is confined. In the computation of uplift the practice of the U.S. Bureau of Reclamation is to compute the uplift as varying linearly from full head water to full tail water, but acting over only the upstream two-thirds of the base.

157. Sliding and Shear. A tendency to slide is caused by the horizontal forces and is resisted by friction between assumed joints on horizontal planes, or between masonry and foundation. Sliding resistance, or shear, can be expressed as

$$f\Sigma W = f(W + V - U \pm X)$$

where W = weight of the structure
V = weight of water on upstream face
U = uplift force
X = sum of all other vertical forces present
f = coefficient of friction

The value of f for rock is frequently used as 0.65. Values from 0.25 to 0.5 are used for earth. The factor of safety against sliding is the ratio between the sliding resistance and the total horizontal force ΣP_h, or $F = f(\Sigma W/\Sigma P_h)$. For masonry dams, values of F vary from 1.0 on rock foundations to 3.0 on earth foundations.

158. Ice Pressure. An estimate of the pressure of ice against a dam can be made from Fig. 68.[1] In using this figure, the following must be known: (1) ice thickness, (2) rate of air-temperature rise, (3) restraint of ice sheet, and (4) extent of exposure to solar radiation. Restraint is of great importance. It is complete if banks are steep and ice is solidly frozen at the restraining edges. If the banks are sloping so that the ice can slide there is no restraint. Other factors to be considered include: (1) currents under the ice, (2) effect of snow on the ice acting as an insulation against temperature change, (3) effect of wind, (4) volumetric expansion of ice during freezing, and (5) effect of changes in reservoir level.

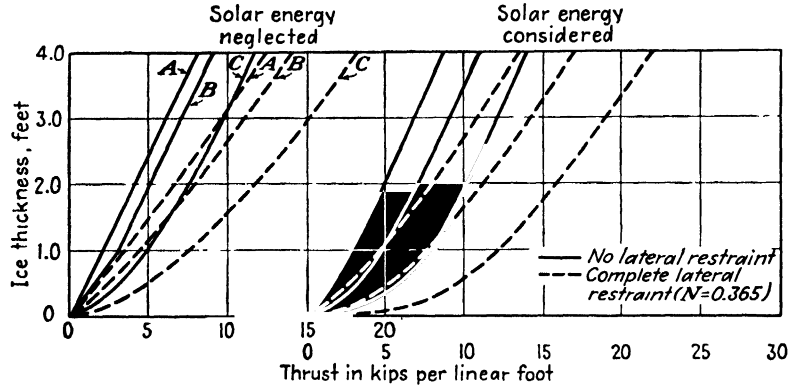

Fig. 68.—Ice thrusts. (*From Edwin Rose, Proc. A.S.C.E., May*, 1946, *p.* 574.)

Conditions used in the figure:

1. Initial air temperature is −40°F.
2. Initial ice temperature varies linearly from −40°F. at the air surface to +32°F. at lower surface.
3. Air temperature changes at rates of +5°F. (curve *A*), +10°F. (curve *B*), and +15°F. (curve *C*) per hour, from −40°F. to +32°F.; then it remains constant at +32°F. until the temperature throughout the ice sheet has risen to about +32°F.
4. Emissivity effects are neglected, *i.e.*, temperature of ice at air surface is assumed same as air temperature itself
5. Thickness of ice sheet remains constant.
6. Diffusivity constant of ice h^2 in square feet per hour, equals 0.0434.
7. Poisson's ratio equals 0.365.
8. Absorption of solar energy is neglected, consideration of this factor being treated subsequently.

While little is known concerning actual ice pressures, practice has been extremely varied. Possibly a pressure of 3,000 to 5,000 lb. per sq. ft. of expected thickness of ice sheet may be conservative. Under the most severe climatic conditions, except in very confined spaces, pressures will seldom exceed 10,000 lb. per linear foot of dam.[1] Pressures due to broken ice, *i.e.*, ice floes, are generally negligible.[2]

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[1] From Rose, Edwin, "Thrust Exerted by Expanding Ice Sheet," with brief bibliography, *Proc. A.S.C.E.*, May, 1946, p. 571.

159. Foundation Reaction. The vertical component of the foundation reaction is equal to the weight of the structure and of the water vertically above it. If the dam acts as a homogeneous unit, as is often the case in a masonry dam, the intensity of the foundation reaction will vary uniformly from toe to heel, as indicated in Fig. 69. It is desirable that there shall be no tension or compression at the upstream edge, or heel, of the dam. To accomplish this the resultant of the external forces on the dam should pass through the edge of the middle third nearer to the downstream portion of the base. The maximum compressive stress, or foundation reaction, will occur at the toe of the dam, as indicated in Fig. 69. Allowable pressures on primary rock vary from 30 tons per sq. ft. down to 10 for good shale. Earth is not suitable for high dams because of its relatively low bearing power. For preliminary designs and estimates for solid-gravity masonry dams a triangular cross section or profile is often assumed. The ratio of base width to height may be taken as about 0.65 for no uplift to 0.91 for full uplift.

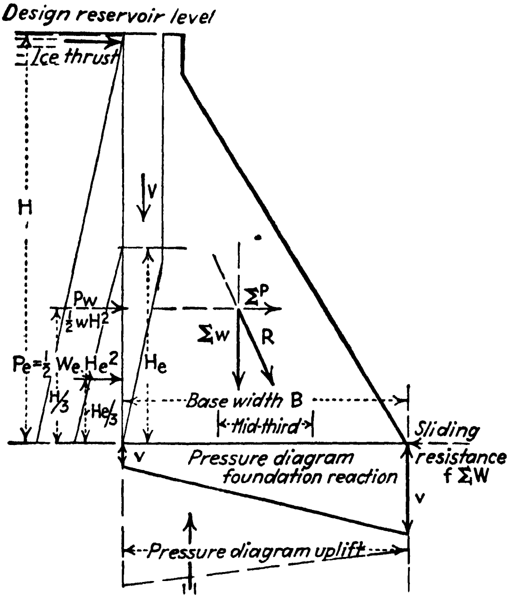

FIG. 69. Principal forces acting on gravity dams.

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[1] See also GISIGER, P. E., "Safeguarding Hydro Plants against the Ice Menace," *Civil Eng.* January, 1947, p. 24.

[2] See also HARRISON, C. L., "Provision for Uplift and for Ice Pressure in Designing Masonry Dams," *Trans. A.S.C.E.*, Vol. 75, 142, 1912.

160. Earthquake Shocks.[1] Stresses[2] resulting from earthquake shocks may occur in a masonry dam due to the acceleration of the mass of the dam and changes in water pressure. The maximum horizontal acceleration due to an earthquake of intensity α is given by the formula

$$a = \alpha g$$

in which g is the acceleration due to gravity (32.2 ft. per sec.^{2}). Vertical accelerations are usually neglected. In the United States a value for α of 0.1 is generally considered adequate on sand and earth foundations, and up to 0.75 for rock foundations. The design of dams in regions subject to earthquakes requires specialized knowledge and experience.

161. Other Forces. Waves act not only to erode the face of a dam but also to create pressure against it. Various empirical formulas for wave pressures have been prepared.[3] Molitor's formula can be expressed as $P = 125h_w^2$, in which P is the total pressure, acting about $\frac{3}{8}h_w$ above the elevation of still-water surface in the reservoir, in pounds; and h_w is the height of wave from trough to crest, in feet, as computed by Stevenson's formula (see Sec. 146).

Wind pressures are usually neglected in dam design.

Subatmospheric pressures may be caused by water falling over the crest of the dam. Since the pressures may become pulsating, their effect on the dam may be dangerous. They should be avoided rather than combatted in the design of the structure. They may be avoided by providing passage for air beneath the crest of the falling water. This procedure is called "aerating the crest."

SPILLWAYS

162. Functions. The function of a spillway is to provide safe passage for water from above the dam to below the dam within the limitations of the available depth of water over the spillway. In the design of a spillway two things must be known: (1) its discharge capacity in terms of its dimensions and the depth of water on it, and (2) the maximum rate of discharge from the reservoir. The capacity of the spillway should be sufficient to pass the greatest flood to be expected from the watershed. Size of floods are discussed in Chap. VI; spillway discharge capacities are discussed in Chap. II and Sec. 165.

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[1] See also WESTERGAARD, H. M., "Water Pressure on Dams during Earthquakes," *Trans. A.S.C.E.*, Vol. 98, p. 418, 1933.

[2] "Water Pressures on Dams during Earthquakes," *Bur. Reclamation, Tech. Mem* 123, 1931; *Trans. A.S.C.E.*, Vol. 98, p. 148, 1933.

[3] See also MOLITOR, D. A., "Wave Pressures on Sea Walls and Breakwaters," *Trans. A.S.C.E.*, Vol. 100, p. 984, 1935.

163. Location. In general, the spillway should be located at a low place in the rim of the reservoir where the distance of fall of water passing the spillway is low. If no suitable low point can be found in the reservoir rim, it may be possible to locate the spillway at the end of the dam and lead the water through a spillway channel on the hillside below the dam. If possible, the location of spillways in the body of a dam should be avoided, particularly in earth dams, because a spillway so located is conducive to erosion at the toe of the dam

164. Types. Spillways may be classified as uncontrolled, automatic, and manual. In an uncontrolled spillway the rate of flow past the spillway varies approximately as the three-halves power of the depth over it. Types of uncontrolled spillways include overflow, chute, and side channel. Automatic spillways automatically provide a minimum of fluctuation of reservoir surface elevation for variations in rates of flow over the spillway. Types of such spillways include siphon, shaft, flash boards, and float-controlled gates. Manual spillways, more commonly called outlet works, control the upstream depth of water by the manipulation of gates or valves which determine the rate of flow of water through the control works. Automatic or manually controlled spillways increase the available storage capacity of a reservoir by minimizing the rise of water level during flood. Water may, therefore, be stored in the reservoir near to flood level during normal flow periods.

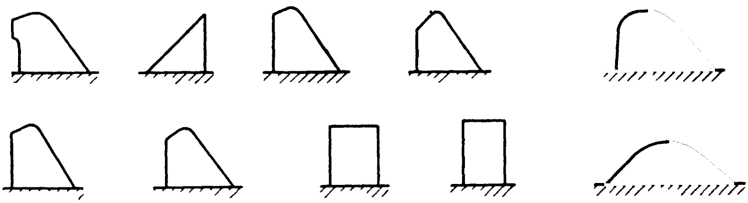

FIG 70 Types of overflow spillways. (*From Proc. A S.C.E., Vol.* 39, *p.* 1513, 1913)

165. Overflow Spillways. Types of such spillways are illustrated in Fig. 70. They are commonly shaped, in transverse section, in the form of an ogee curve or as a broad-crested weir The rate of discharge over them can be expressed as $Q = Clh^{3/2}$, in which Q is the rate of flow over the spillway for a length l along the spillway, h is the depth of water on the spillway measured vertically from the top of the spillway to the elevation of the surface of the water in the reservoir, and C is a coefficient, values of which are discussed in Sec. 33.[1]

The downstream face of the spillway must be designed to avoid scour. The production of turbulence in the stream before the water has left the apron serves to dissipate some of the scouring energy. A hydraulic jump is an effective means for absorbing energy. Other schemes for the purpose include sills dentated in plan, as shown in sketch,

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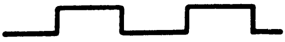
[1] See also OFFITZEROFF, A S., "Model Studies of Overflow Spillway Sections," *Civil Eng.*, August, 1940, p. 523.



which direct streams of falling water against each other; protruding ledges and obstructions in the face of the dam; baffle piers; and stilling pools.

166. Spillway Channels. Water may be conducted away from spillways to the main stream bed in spillway channels, or it may fall directly into the stream bed. Types of spillway channels include flumes, tunnels, chutes, and side channels. Flumes and tunnels may be constructed as in aqueducts. Where local topography is favorable, it may be possible to construct a steeply sloping channel to carry the falling water at a higher, but less destructive, velocity than would result either from a direct fall or by following an ogee curve. The term "chute spillway" refers, in general, to a spillway not in the face of the dam. Instead, the overflow discharges to the water below the dam through a lined trench or flume. The length of the spillway is in a position approximately normal to the axis of the chute. The bottom of the channel may be roughened to increase the rate of absorption of energy.

Where water falls over a spillway into a chute that is parallel to the length of the spillway, the chute may be known as a "side-flow channel." The hydraulics of side-flow channels have been analyzed by Camp,[1] who has formulated the flow in a rectangular flat-bottom channel with constant slope as

$$H_0 = \sqrt{d^2 + \frac{2Q^2}{gb^2d} - Sx\bar{d} + \frac{fxQ^2}{12gb^2\bar{R}\bar{d}}}$$

where H_0 = depth of water at upper end of spillway
d = depth at any point
Q = rate of flow
b = width of channel
S = bottom slope
x = distance from upper end of channel to point where depth is d
$\bar{d}$ = average depth throughout distance x
f = Weisbach-Darcy friction factor
$\bar{R}$ = average hydraulic radius throughout distance x

Where velocities are high in spillway channels or at their junction with the stream bed, special attention must be given to prevent erosion.

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[1] CAMP, T. R., "Lateral Spillway Channels," *Trans. A.S.C.E.*, Vol. 105, p. 606, 1940.

167. Siphon Spillways. A section through a siphon spillway is shown in Fig. 71. For the lowest flows the spillway acts as a covered overflow spillway, but when the rate of flow is sufficient to fill the channel a siphon is formed and the effective discharge head is measured vertically between the elevation of the water surface in the reservoir and either the center of the discharge opening below the spillway, or the surface of the water in the tailrace, whichever is the higher. Because of practical limitations the effective value of this head h cannot exceed about 24 ft.[1]

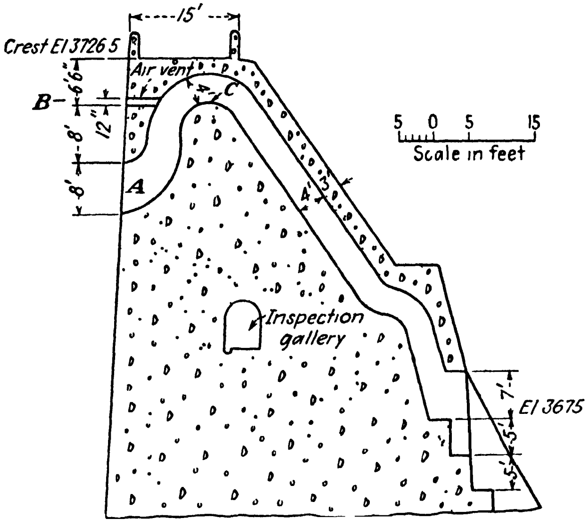

FIG. 71 —Siphon spillway section, O'Shaughnessy Dam, city and county of San Francisco, Calif.

The rate of discharge can be computed from the expression $Q = kA\sqrt{2gh}$, in which A is the cross-sectional area of the siphon at the throat or the narrowest portion and k is a constant usually taken at about 0.65.

Advantages of siphon spillways include (1) close regulation of reservoir surface, (2) high capacity within relatively small space as compared with overflow spillways, (3) automatic action, (4) wide range or operating head, and (5) no moving parts. On the other hand their disadvantages include (1) serious danger due to clogging, (2) destructive vibrations due to alternate breaking and forming of siphonic action, and (3) relatively high cost of construction.

168. Shaft Spillways.[2] A section through a shaft spillway is shown in Fig. 72. Such spillways possess the advantages and disadvantages of the siphon spillway. Where local topographical or other conditions are favorable, they may be the least expensive type of spillway to install. However, if the fall is high, erosion from cavitation and high velocities may be serious.

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[1] See also ROCK, ELMER, "Design of High-head Siphon Spillway," *Trans A.S.C.E.*, Vol. 105, p 1050, 1940.

[2] See also TRAHERN, J. W., "Design of Circular Spillway Modified to Cure Pulsating Discharge," *Eng News-Record*, July 1, 1943, p 77

169. Flashboards and Tilting Gates. Flashboards are designed to collapse when the depth of water on them exceeds a predetermined maximum. Upon the collapse of the flashboard the effective capacity of the spillway is increased without raising the elevation of the water in the reservoir. After the passage of the flood the flashboard may be replaced manually or automatically. Modifications of the flashboard principle include counterbalanced or tilting gates that automatically tilt under high heads and restore themselves under low heads.

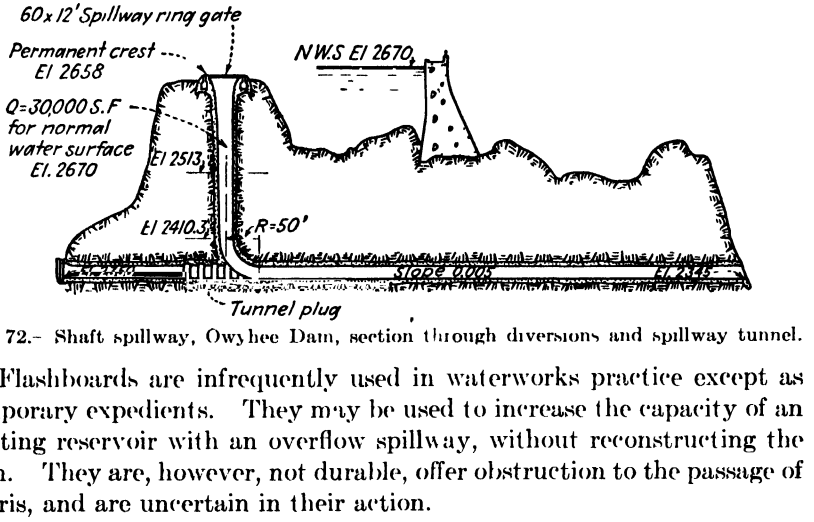

FIG. 72.– Shaft spillway, Owyhee Dam, section through diversions and spillway tunnel.

Flashboards are infrequently used in waterworks practice except as temporary expedients. They may be used to increase the capacity of an existing reservoir with an overflow spillway, without reconstructing the dam. They are, however, not durable, offer obstruction to the passage of debris, and are uncertain in their action.

170. Outlet Works. Outlet devices are provided to control the release of water from a reservoir for any purpose. An intake, as discussed in Chap. VIII, is an outlet from the reservoir. Outlet works may be used also for releasing excess water, for emptying the reservoir, or for control of water supplied to downstream owners of prior water rights.

The location, capacity, structural features, and safety of the outlet works are interrelated and may depend also on the purpose of the works. For most purposes, other than to supplement spillway capacity or to control maximum reservoir elevation, outlets are placed low in the reservoir so that its full capacity may be available, or it may be completely drained. Locations at intermediate elevations may be desirable in deep reservoirs to permit the withdrawal of water of the best quality in the reservoir.

Control devices and outlet works include overflow-spillway or crest-control devices and gates or valves. Types of crest-control gates, in addition to flashboards, include radial or Tainter gates, drum gates, and sliding gates.

A cross section of a radial or Tainter gate is shown in Fig. 73. The face, or skin plate, is bent to a circle the radius of which is the distance between the plate and the center of the horizontal pin on which the gate turns. The face plate is supported on a steel frame. Hoisting ropes are fastened to the bottom of the gate and lead to spindles on a platform above the gate. When the gates are raised, the hoisting ropes are wound around the spindles. The spindles are turned by a hand-operated or motor-driven hoist fitted with the necessary reduction gearing to suit the ratio between the power and the load. Radial gates do not ordinarily operate automatically.

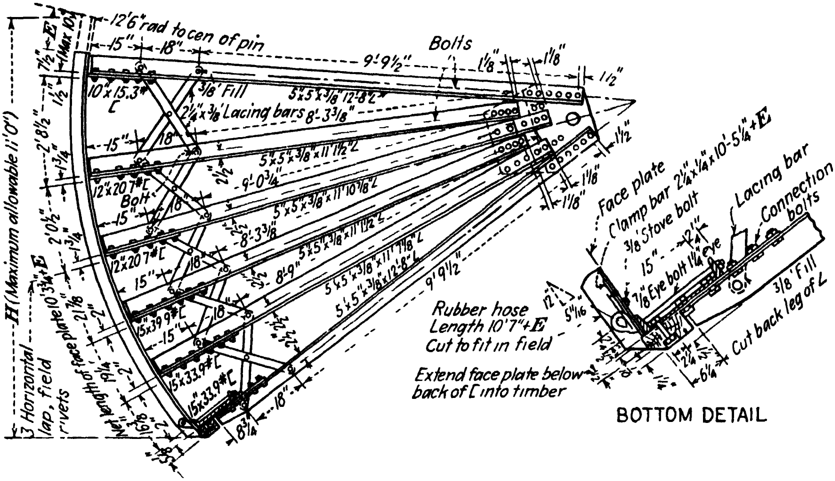

Fig. 73. Detail of an 18-ft. radial gate. (*Courtesy of U.S. Bureau of Reclamation.*)

The drum gate, a cross section of which is shown in Fig. 74, is an automatic type designed to hold the level of the reservoir constant. The drum is built of plate steel properly braced and is watertight. It floats on water contained in the drum-gate chamber and rotates about a hinge at the upstream edge. The level of the water in the chamber is controlled by a float which operates a balanced valve. When the water in the reservoir tends to rise above normal, the float opens the balanced valve which permits part of the water in the chamber to escape. The gate, floating on the water, lowers with it until the necessary additional spillway capacity is established. When the flow over the spillway becomes constant, the balanced valve is closed, and the water in the chamber and the gate are held at the required height. When the water in the reservoir drops below normal, the balanced valve admits water into the chamber, and the gate rises. The drum gate is used when close regulation of reservoir level is necessary.

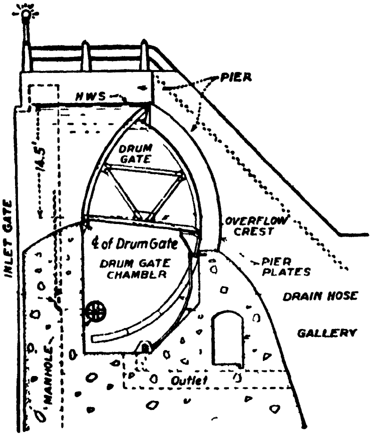

FIG. 74.—Cross section of drum gate and chamber.

Sliding gates consist of a deck or skin plate resting on a structural frame of beams and girders which slides in grooves or on tracks that are installed in piers and abutments. Unless provided with caterpillars or rollers to overcome friction, an excessive amount of power is required to operate them.

Leaf, or flap, gates are hinged at the bottom and are raised and lowered by means of a crane. They are suitable for low diversion dams. Figure 75 shows a section of a flap gate.

The bear-trap gate is an automatic type used to regulate the level of the reservoir. It consists of two hinged leaves which are raised or lowered by a difference in water pressure on the upper and lower surfaces of the leaves. It is rarely used in waterworks practice.

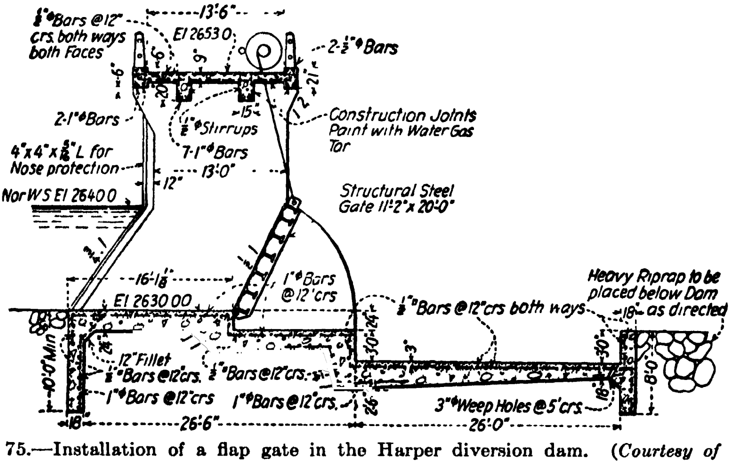

FIG. 75.—Installation of a flap gate in the Harper diversion dam. (*Courtesy of U.S. Bureau of Reclamation.*)

The sector rolling gate consists of a steel drum the cross section of which is in the shape of a sector of a circle. At each end a circular casting having gear teeth on its outside face is fastened to the circular part of the drum. The gear teeth engage the teeth of a sloping rack fastened to the piers. The lifting force, applied by a hoist on a platform above the gate, causes the gate to roll on the gearing and lifts it from a concrete chamber provided in the top portion of the masonry. Sector rolling gates are adaptable for heights of 10 to 20 ft. and in lengths from 50 to 100 ft.

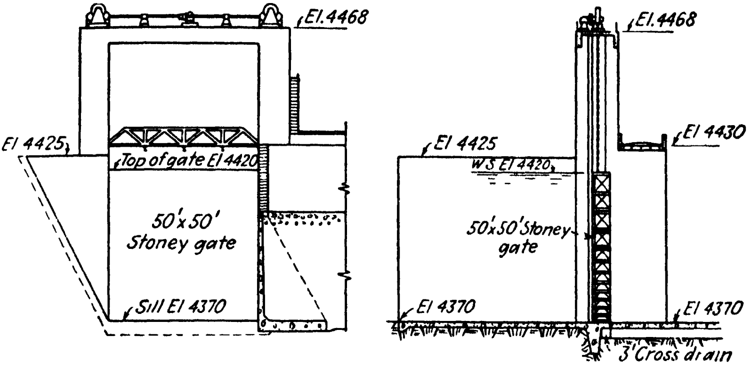

FIG. 76.—Elevation and section of Stoney gate at Guernsey dam.

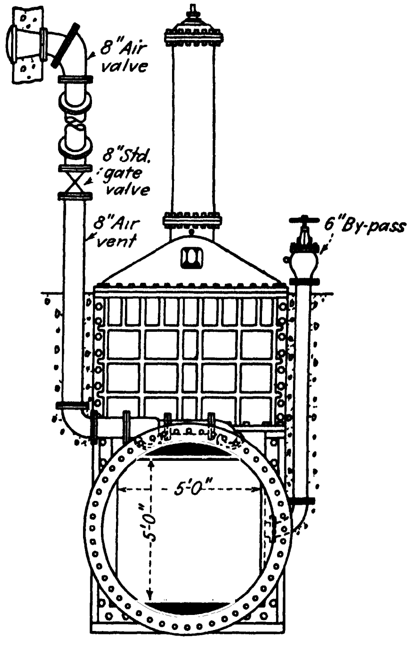

FIG. 77.—Hydraulically operated high-pressure slide gate.

Outlet Gates and Valves. The purpose of outlet gates and valves is to control the quantity of water drawn from the reservoir. For waterworks they are, respectively, of two general types: slide gates and needle valves.

A Stoney gate, as illustrated in Fig. 76, is a special form of large slide gate, usually provided with rollers or caterpillars to minimize friction and power for operation. A slide gate usually consists of a rectangular gate or leaf, properly mounted, which slides over an opening in a pipe or conduit. The leaf is moved by means of a screw or by hydraulic pressure against a piston contained in a cylinder, mounted over the leaf. Figure 77 shows a hydraulically operated slide gate. Slide gates are considered as low-pressure gates when they operate against heads of less than about 60 ft. Gates operating against greater heads are ordinarily classed as high-pressure gates. Slide gates are not suitable for use for heads greater than about 250 ft., but new types developed by the Bureau of Reclamation and known as ring-follower and Paradox may be used for heads up to 600 ft.

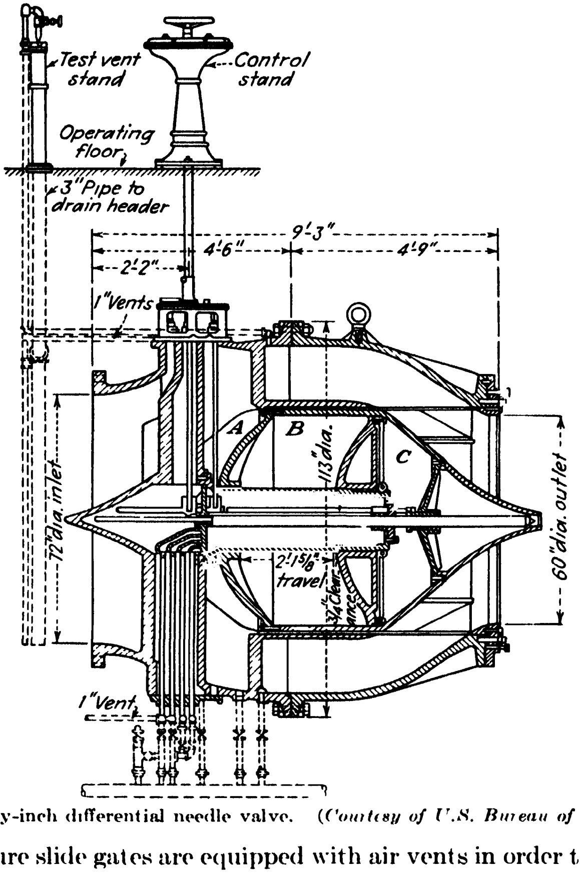

FIG. 78.—Sixty-inch differential needle valve. (*Courtesy of U.S. Bureau of Reclamation.*)

High-pressure slide gates are equipped with air vents in order to eliminate pitting and erosive effects which are liable to occur in the outlet channel as a result of pressures less than atmospheric. Slide gates, if properly designed, manufactured, and installed, are satisfactory in operation and have little to get out of repair. They are slow in operation, especially under high heads.

A needle valve consists of a piston or needle that slides in an internal cylinder, closed at the upstream end, installed within a truncated ellipsoidal casting. The cylinder is held in place by ribs connecting it with the outside shell. The valve is opened and closed by the manipulation of pressures in the chambers A, B, and C (Fig. 78). Figure 79 shows a cross section of an outlet works controlled by a needle valve.

Outlet-control devices should be placed on the upstream end of conduits leading through the dam. It is undesirable to pass conduits through earth dams because of the possibility of breakage resulting from uneven settling of the embankment or to "piping" along the outside of the conduit.

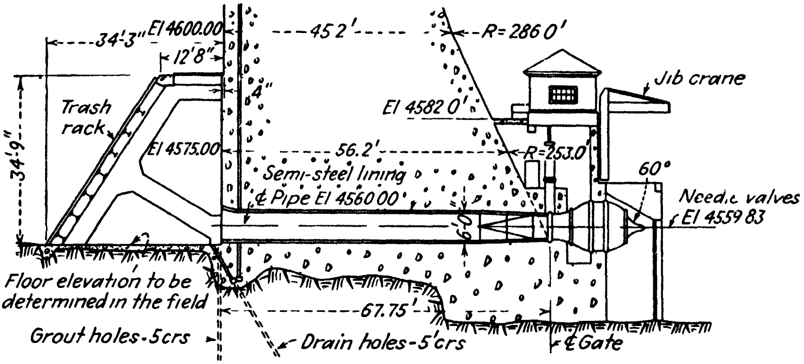

FIG. 79.—Cross section through an outlet works, showing needle-valve installation.

171. Logways and Fishways. Logways are provided to facilitate the passage of logs over dams located on streams that are used for logging operations. Logways are usually built 6 to 30 ft. wide and require 25 to 50 c.f.s. for their operation. Flow into the logway is best controlled by a radial gate. Mechanical devices are sometimes used for passing the logs by the dams to avoid a loss of water. Logways may also be used to remove floating debris from the reservoir.

A fishway, or fish ladder, is a device to permit fish to pass upstream or downstream over or around a dam. The laws of some states require the installation of such devices.

The design of the fish ladder will depend on the habits of the species of fish that live in the stream and the requirements of the body appointed to administer the laws of the particular state. Jumping fish usually require a pool ladder which consists of a series of dams and pools built in an inclined channel. Water flows from 6 in. to 2 ft. deep over each dam into the pool below it, thus providing a waterway from the reservoir to the stream. Fishways may also be formed by constructing an artificial inclined channel without dams. The latter type ordinarily requires a greater amount of water for its operation. The height to which a ladder can be built depends on the height to which certain types of fish will ascend a ladder. The U.S. Bureau of Fisheries does not recommend heights greater than 40 ft.

CHAPTER VIII

INTAKES

172. Types of Intakes.[1] A waterworks intake is a device or structure placed in a surface-water source to permit the withdrawal of water from this source and its discharge into an intake conduit through which it will flow into the waterworks system. Types of intake structures consist of intake towers, submerged intakes, intake pipes, or conduits, movable intakes, and shore intakes. Intake structures over the inlet ends of

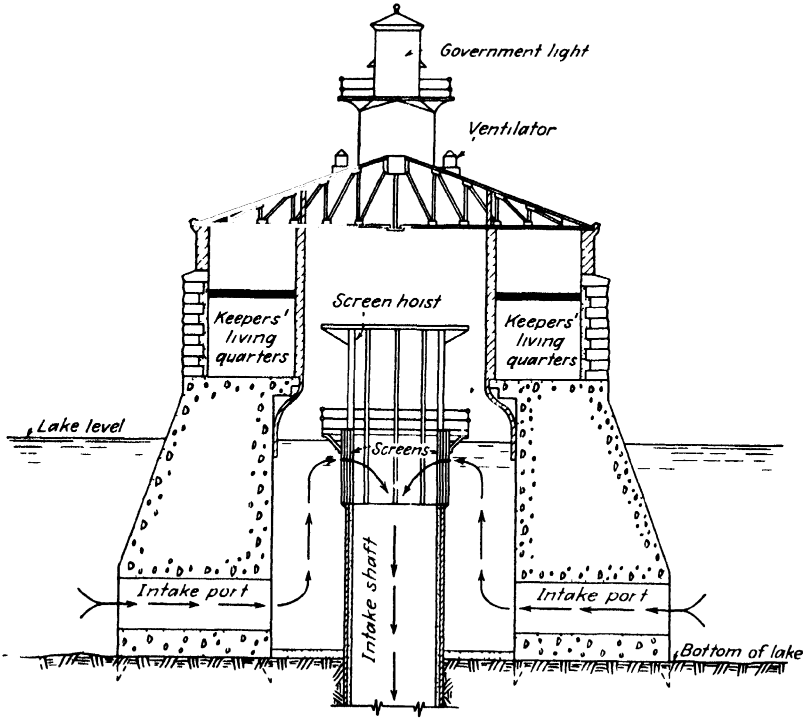

FIG. 80.—Typical Chicago intake crib. *(From C B Burdick, Jour A W.W A., March, 1946, p. 317)*

intake conduits are necessary to protect against wave action, floods, stoppage, navigation, ice, pollution, and other interference with the proper functioning of the intake.

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[1] See also BURDICK, C. B., "Water Works Intakes," *Jour. A.W.W.A.*, March, 1946, p. 315; and McDONALD, N. G., "Water Works Intakes," *Jour. A.W.W.A*, April, 1940, p. 661.

Intake towers are used for large waterworks drawing water from lakes, reservoirs, and rivers in which there is either, or both, a wide fluctuation in water level or the desire to draw water at a depth that will give water of the best quality, to avoid clogging or for other reasons. Fluctuations of as much as 80 ft. are experienced in the Ohio River. A wet intake tower is filled with water to the level of the source of supply. A section through such a tower, typical of those used on the Great Lakes, is shown in Fig. 80. A dry intake tower has no water inside it other than in the intake pipes. Water is admitted through the ports connected to the intake

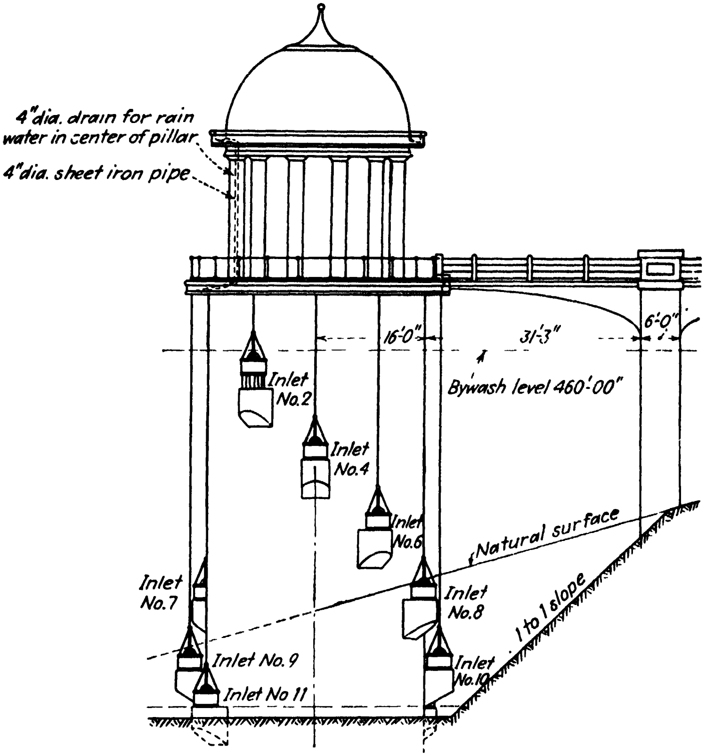
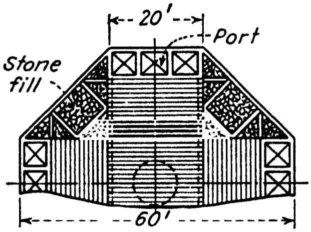


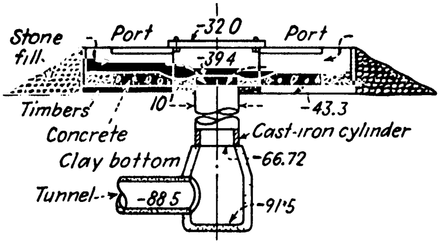
FIG. 81.—Outlet tower of Maroondah Reservoir, Australia. This reservoir supplies about a half million people and provision is made to take water through 12 different inlets. These inlets are in pairs located diagonally opposite one another and thus provide for draft at six different levels in a total depth of approximately 55 ft. Each inlet may be shut off by a sleeve valve held in place by a cable. Thus, the flow of water may be cut off so that, if necessary, the valves inside the tower can be removed for repairs. These are raised or lowered by a counterweight device which may be submerged. (*From E. G. Ritchie, Water Works Eng., Jan.* 22, 1947, *p.* 72.)

pipes which convey it from the tower under pressure. The interior of the tower is thus made accessible for inspection and operation. Wet towers are less costly to construct. They are not subject to flotation, and certain other stresses need not be considered. Towers in lakes and in some rivers are usually located some distance from the shore line. An Australian reservoir tower is illustrated in Fig. 81.[1]

Submerged intakes, an example of which is shown in Fig. 82, are constructed entirely under water and have such advantages over exposed structures as lower cost, no obstruction to navigation and but little obstruction to the flow of the river, little danger from floating material, and a minimum of troubles from ice. This type of structure is commonly used for intakes of moderate size on the Great Lakes and for the majority of other installations.



PLAN



CROSS-SECTION
ON CENTER LINE

FIG. 82. Submerged crib in Lake Michigan at Gary, Ind., adaptable to conditions where adequate depth is available. (*From C. B. Burdick, Jour. A.W.W.A., March,* 1946, *p.* 318.)

An intake pipe, such as is shown in Fig. 83, offers a satisfactory conduit under favorable conditions. It should be used, where possible, to minimize cost.

Movable intakes, many of which are to be found along the Missouri River, consist of a pump mounted on rails at right angles to the river bank. As the river level changes, the pump is run up or down the bank and the discharge from the pump is connected through tees at an appropriate level on a manifold pipe that extends down to the low-water level. This type of intake is used where shifting shoals, lack of foundation in the stream bed, or other conditions preclude the erection of an intake structure in the stream.

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[1] See RITCHIE, E. G., "Reservoir Outlet Towers at Melbourne," *Water Works Eng.*, Jan. 22, 1947, p. 72.

Shore intakes, an example of which is shown in Fig. 84, may be suitable for industrial plants where water quality is not a primary consideration or for public water supplies where water quality and other conditions permit. An outstanding example of a successful shore intake for a public water supply exists at the South Side Water Filtration Plant in Chicago. Shore intakes will probably be less costly than towers or cribs.

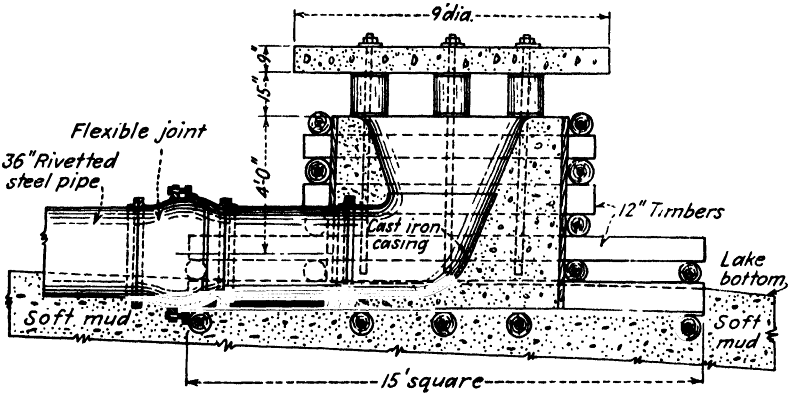

FIG. 83.— An intake crib designed for use in a bay. (*From N. G. McDonald, Jour. A.W.W.A., April,* 1940, *p.* 666.)

173. Location of Intakes. Conditions to be considered in the location of an intake include (1) the location of the best quality of water available; (2) currents that might threaten the safety of the intake structure; (3) navigation channels, which should be avoided; (4) ice floes and other ice difficulties; (5) formation of shoals and bars; and (6) fetch of the wind affecting the height of waves. Conditions affecting the quality of water will include currents due to wind, temperature, seasonal turnover, and other causes that will bring water of unsuitable quality to the intake. Channels with high currents carrying floating debris and ice are hazardous to the safety of the structure. Navigation channels add the danger of pollution from toilets and other refuse discharging from ships.

Ice floes are hazardous to the superstructure of an intake, owing to pressures exerted on it, and to intake ports both shallow and deep, since ice floes may push down surface ice to clog ports 20 to 30 ft. below the water surface. Waves are hazardous to the superstructure of the intake, and they stir up mud and silt from the bottom in such quantity as to affect the quality of the water.

A study of the currents in a lake or river should be made before selecting the location of an intake in order to assure water of the best quality and the avoidance of polluted water. Such studies are made by observing the progress of both surface and submerged floats.

In bodies of water having wide fluctuations in level, the intake should be located so that, at the lowest water stage, one submerged inlet is entirely under water and the lowest port in an exposed intake is submerged. An intake in an impounding reservoir is usually placed in the deepest part of the reservoir, which ordinarily is near the dam. This will render the full capacity of the reservoir available and will usually protect the intake from sediment in the reservoir. Intakes in streams with a steep bottom gradient should, where possible, be placed at sufficient distance upstream to supply water to the city by gravity.

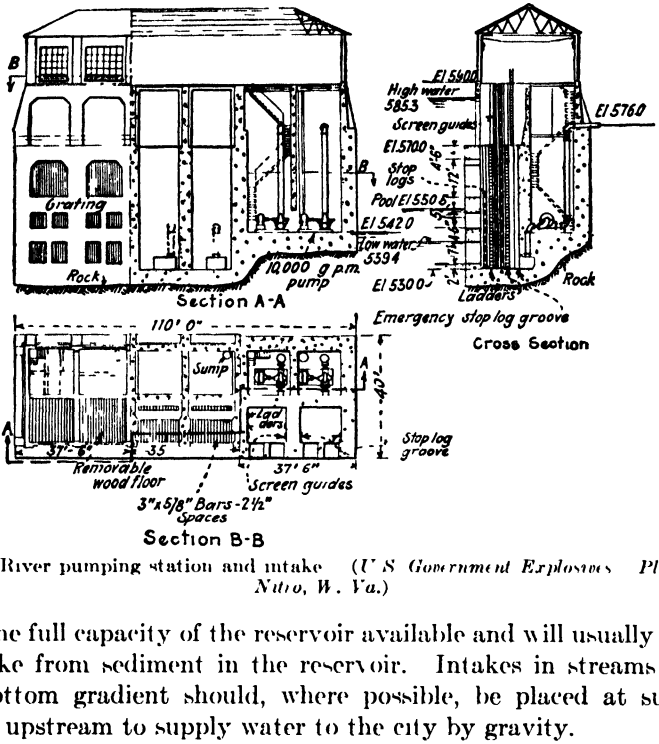

FIG. 84. River pumping station and intake (*U.S. Government Explosives Plant C, at Nitro, W. Va.*)

The final decision on the location of an intake should be based on a consideration of the factors of quality of water, safety of supply, and cost.

174. Design of Intakes. External forces to be resisted by intakes include flotation, waves and currents, ice pressures, blows from floating and submerged objects, and shifting shoals. The magnitude of such forces is known only approximately; therefore a generous factor of safety must be allowed.

Flotation can be computed on the basis of the weight of the volume of water displaced. Since air may be entrapped in intake conduits and in intake towers, and the towers may be pumped dry, a generous allowance should be made for flotation and to resist collapse against external hydrostatic pressure. Flotation is overcome by the construction of massive masonry structures; by weighting down submerged cribwork with broken stone, as shown in Fig. 82; by filling the annular space between two concentric cylindrical steel shells with stone or concrete, the cylinders being sunk to the bottom and supported thereon.

Little is known about pressures from waves and currents. Waves ten feet high may cause pressures of 1,800 to 3,000 lb. per sq. ft., with the maximum at mean water level.[1] Wind pressures up to 20 lb. per sq. ft. should be allowed against the portion of a tower exposed above water level.

The force of blows by floating or submerged objects should not exceed those due to moving ice. Intakes in or near navigable channels should be protected by clusters of piles, or other defense, against collision by moving objects.

Undermining of foundations due to water currents or overturning pressures due to deposits of silt against one side of an intake structure are to be avoided rather than combated. Where these conditions are unavoidable, estimates of the magnitude of the forces involved must be made on the basis of tests at the site of the intake or from knowledge of the behavior of similar structures under similar conditions.

The entrance of large objects into the intake pipe is prevented by the use of a coarse screen or by the obstructions offered by the small openings in the cribwork or riprap placed around the intake pipe. Fine screens for the exclusion of small fish and other small objects should be placed at some accessible point, as at the suction or wet well at the pumping station where the screens can be easily inspected and cleaned. The location of the screens in an inaccessible position in a remote intake structure is undesirable. The area of the openings in the intake crib should be sufficient to prevent an entrance velocity higher than about 30 f.p.m., in order to avoid the carrying of settleable matter into the intake pipe.

An intake tower and pumping station are illustrated in Fig. 84. Intake ports should be placed at various elevations in the tower so that water of the best quality may be taken. They should also be placed so that if one or more of the ports is blocked another can be opened. It should be possible to open or close the ports in the intake from the operating floor, as indicated in Fig. 84. Where the intake tower is in a river in which ice troubles may be encountered, the upstream side of the tower should be protected and strengthened to resist ice pressure. In general, the intake ports should be placed in the downstream face of the tower, and they should be protected against blocking by ice or other floating material.

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[1] See also "American Civil Engineers' Handbook," 5th ed., p. 1108, John Wiley & Sons, Inc., New York, 1930.

Submerged ports should be designed and controlled to prevent air from entering the suction pipe. The difficulty can be minimized by maintaining an entrance velocity not greater than 2 f.p.s., preferably much less than this, and maintaining a depth of water over the port of at least three diameters of the port opening.

175. Screens. Screens or grids with parallel bars, preferably removable, may be placed over the ports, as indicated in Fig. 84. Mesh screens are undesirable because of the difficulty of cleaning them. Screens are used to prevent large floating objects from entering the intake or gaining access to the conduit. Screens should not be used as a purification device for the mechanical straining of suspended matter. This can probably be done more effectively and more economically at purification plants ashore. Hence, the openings between bars may not be less than 1 to 2 in. The velocity through a screen should not exceed about 30 f.p.m. and is preferably much less than this. In some installations screens are used only occasionally under suitable conditions; in others, automatically and mechanically cleaned screens are used. An electrically charged screen has been devised to keep fish away.

176. Difficulties with Ice.[1] Surface ice, anchor ice, frazil ice, and slush ice are forms in which ice may be troublesome at an intake. Surface ice floes, many miles in extent, push against tower intakes in the Great Lakes and pile ice against them from the bottom to heights of 25 ft.[2] above the lake surface. Ice pressures shake the structures and clog the ports, necessitating the presence of a maintenance crew living on the intake structure.

Anchor ice is formed beneath the water on the surface of objects that radiate heat rapidly. It forms most readily on dark metallic surfaces such as valves and gratings. It is usually formed at night in clear cold weather when the surface water is cold but is too turbulent to freeze. A sheet of surface ice will prevent the formation of anchor ice or of frazil ice. It may be possible to minimize troubles from anchor ice by locating the intake in a quiet locality where the surface will freeze readily, or by constructing some form of temporary surface protection around the proposed intake.

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[1] See also BARNES, H. T., "Ice Engineering," Renouf Publishing Co., Montreal, 1928.

[2] See BURDICK, C. B., "Water Works Intakes," *Jour. A.W.W.A.*, March, 1946, p. 415; and GISIGER, P. E., "Safeguarding Hydro Plants against the Ice Menace," *Civil Eng.*, January, 1940, p. 24.

Frazil ice, sometimes called needle ice, consists of submerged ice crystals that have been formed as anchor ice and have broken loose, or are ice crystals that have formed about small particles suspended in water which serve as neuclei. Slush ice is formed of a soft mushy mass of ice crystals ground from other forms of ice.

Under conditions favorable for its formation each type of ice may cause difficulties at waterworks intakes.

Expedients used for the removal of ice from intake ports include a vigorous reversal of the flow of water through the port, steam or electrical heating devices, compressed air, pike poles, chains, and explosives. Divers have been employed under urgent conditions, but the service is hazardous and should be avoided. Provision for vigorous reversal of flow should be made when the intake is installed. This can be done by draining an elevated storage tank into the intake conduit, or discharging the flow from the main pumps into it the pumps drawing their water from storage on the distribution system. The height of the curb on the intake well above the elevation of the water in the intake should be sufficient to provide the head necessary to force water backward through the intake conduit.

177. Intake Conduit and Intake Well. The conduit conveying water from the intake should lead to a suction well in or near the pumping station. Either a pipe lying on or buried in the bottom of the body of water or deep tunnels may be used as intake conduits. For conduits up to about 12 in. in diameter, standard bell-and-spigot cast-iron pipe may be used, occasionally with flexible joints. Larger conduits may be of steel or concrete. A tunnel, although more expensive, makes the safest and most satisfactory conduit.

The capacity of the conduit and the depth of the suction well should be such that the intake ports to the pumps will not draw air. A velocity of 2 to 3 f.p.s. in the intake conduit, with a lower velocity through the ports, will give satisfactory performance. The horizontal cross section of the suction well should be three to five times the vertical cross-sectional area of the intake conduit. Even with conduit and suction well of adequate capacity it might be possible, by starting the pumps too fast, to draw water out of the suction well and expose the pump suction ports. Pumps should be started gradually to avoid drawdown in the suction well and stopped slowly to prevent surges. The intake well acts as a surge tank on the intake conduit, minimizing the surge. The factors involved are mathematically related, as shown in Sec. 194.

The intake conduit should be laid on a continuously rising or falling grade to avoid accumulation of air or gas, pockets of which would otherwise restrict the capacity of the conduit. Where air traps are unavoidable, provision must be made to allow gas to be drawn off from them. Where pipes are used, they must be weighted down to prevent flotation.

Because of the dependence of a waterworks on its intake, intake conduit, and suction pit, it is desirable that most of the works be in duplicate and, where possible, two or more widely separated intakes may be provided.

CHAPTER IX

AQUEDUCTS

178. Definition.[1] An aqueduct is a conduit designed to convey water from a source to a point, usually a reservoir, where distribution begins. An aqueduct may include canals, flumes, pipe lines, siphons, tunnels, or other channels, either open or covered, flowing at atmospheric pressure or otherwise. A typical profile of an aqueduct over rough country is shown in Fig. 85. The choice between available types of conduits depends on such factors as topography, pressure head available, quality of water, conditions of construction, and economy. The construction of such aqueducts as the Colorado River Aqueduct for Los Angeles, the Delaware Aqueduct and the Ashokan Aqueduct for New York City, the Hetch Hetchy Aqueduct for San Francisco, and the Quabbin Aqueduct for Boston have involved spectacular engineering works and the expenditure of large sums of money.

An important consideration in the design of an aqueduct is the prevention of pollution of the water in the conduit. Among the most common sources of pollution to be guarded against may be included infiltration or surface runoff from the surrounding ground and the direct pollution of water in uncovered conduits.

179. Canals. A canal is an open conduit, either covered or uncovered, designed to convey water. An open conduit is a channel conveying water in which the hydraulic grade line lies in the surface of the water. A canal supported on or above the surface of the ground may be called a flume. An underground canal is a tunnel. Conditions favorable to the construction of a canal include topography permitting construction on the hydraulic grade line, impervious material easily excavated and not easily eroded or choked by plant growth, and surroundings free from pollution. A combination of all favorable conditions is unusual. Objections to the use of canals for conveying public water supplies include the hazard of pollution of the water, high seepage and evaporating losses, difficulties of handling cross drainage, ice, danger from burrowing animals, and high maintenance costs.

The location of the route of a canal along a hydraulic grade line will require a study on a topographic map to balance cuts and fills, minimize crossings of streams and ravines, avoid tunnel constructions, and avoid the objections listed above.

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[1] See also WATSON, JOHN D., "Notes on the Construction of Aqueducts," *Engineer*, Vol. 168, pp. 344 and 368, 1939.

A semicircle is the most economical shape of cross section hydraulically, as it gives a maximum hydraulic radius for a given cross-sectional area. A trapezoidal channel is easier to construct and to maintain. A trapezoid in which a semicircle, with its center in the water surface, can be inscribed gives the best hydraulic characteristics for a trapezoidal cross section. The steepness of the side slopes of the trapezoid, in unlined canals, depends on the angle of repose of the material that forms the banks. In earth channels the slopes vary from $2\frac{1}{2}$ horizontal to 1 vertical in sand, to $1\frac{1}{4}$ to 1 in stiff clay. In rock the sides may be vertical.

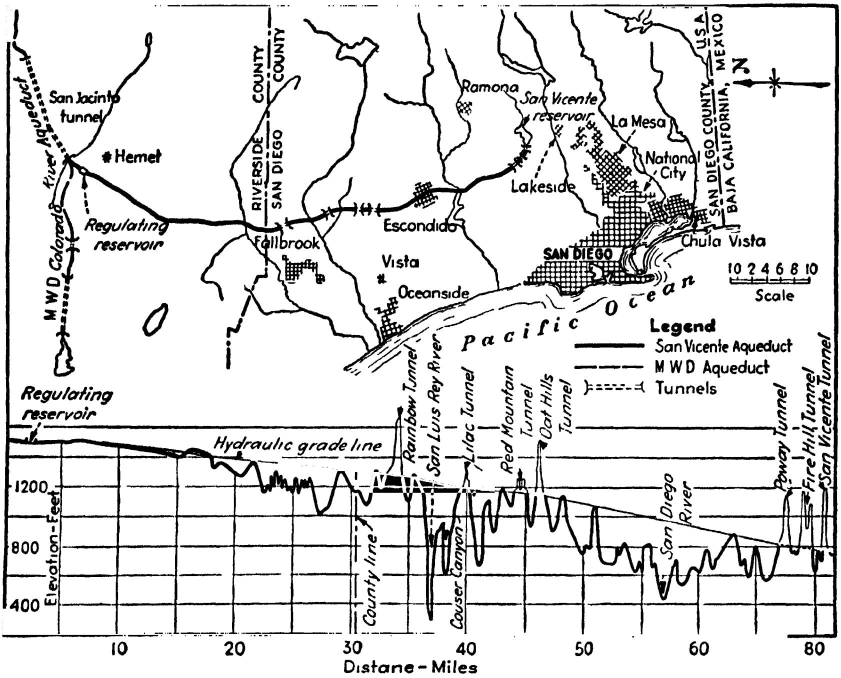
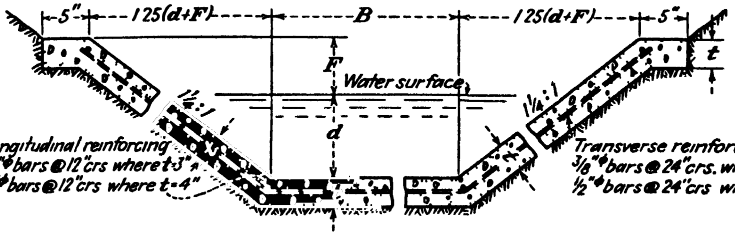


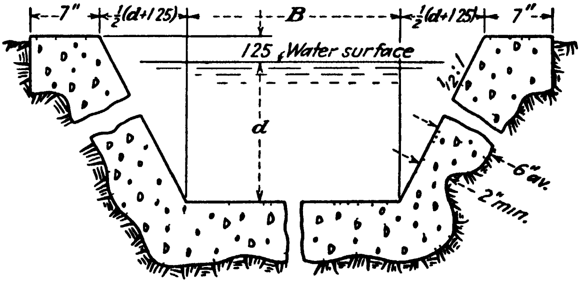
FIG. 85.—Aqueduct over rough topography, for San Diego (*From Eng News-Record, Mar 6, 1947, p 65*)

The use of lining in a canal has the effect of permitting higher velocities and thus reducing the required cross section. The surface of the lining offers less resistance than earth to the flow. Hence, friction losses are reduced. Linings also reduce seepage losses and maintenance costs. The general features of canal linings are similar to those of reservoir linings discussed in Sec. 370, except that construction joints are usually omitted. Figure 86 shows details of lined canals in earth and rock. The figure also shows a method by which a number of different cross sections may be shown on contract drawings. The location of a particular section may be shown on the profile, and a table is provided to show dimensions.

The need for basing the selection of the cross section of a channel on its hydraulic properties is, perhaps, overbalanced in many cases by other considerations such as economy, facility of construction, structural characteristics, and practicability. Some typical cross sections of waterworks canals are shown in Fig 86.


5"
125(d+F)
B
125(d+F)
5"
t
F
Water surface
d
$1\frac{1}{4}$:1
$1\frac{1}{4}$:1
Longitudinal reinforcing
3/8"⌀ bars @ 12" crs where t-3"
1/2"⌀ bars @ 12" crs where t-4"
Transverse reinforcing
3/8"⌀ bars @ 24" crs. where t-3"
1/2"⌀ bars @ 24" crs where t-4"

LINED EARTH SECTION



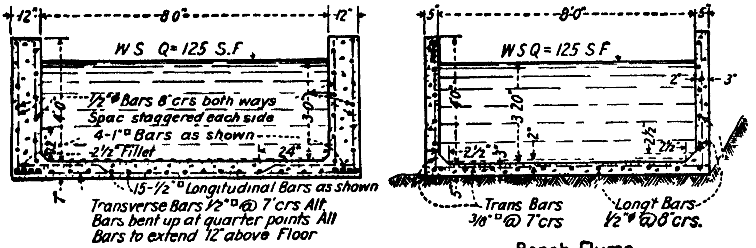
LINED ROCK SECTION

Fig. 86.—Details of lined canals (*Courtesy of U S Bureau of Reclamation*)

Flumes are frequently constructed with walls of concrete, steel, or timber, either self-supporting or supported with the aid of other structural members. Sections of two concrete flumes are shown in Fig. 87. Metal flumes may consist of sheet-metal plates bent in the form of a semicircle, supported by stringers, usually wood, to reduce lateral stresses on the piers or bents. The weight of the flume and water is transferred to carrier beams by steel rods bent to semicircular shape. The carrier beams are supported by stringers and serve also to prevent the inward collapse of the thin metal.

Velocities of flow in canals and flumes should be high enough to consume the available head without eroding the banks or exceeding the critical velocity at which the energy (depth plus velocity head) is a minimum. At higher than critical velocities jumps, bores, and standing waves may occur and result in overtopping of the canal banks or other undesirable feature.

In general, flumes are a type of waterworks aqueduct to be avoided. Difficulties met in their design and maintenance include prevention of leakage, provision for expansion and contraction, variations in the invert gradient caused by unequal settling of the supports, exposure to extremes of temperature and wind, and overflow due to clogging or surge.



Suspended Flume

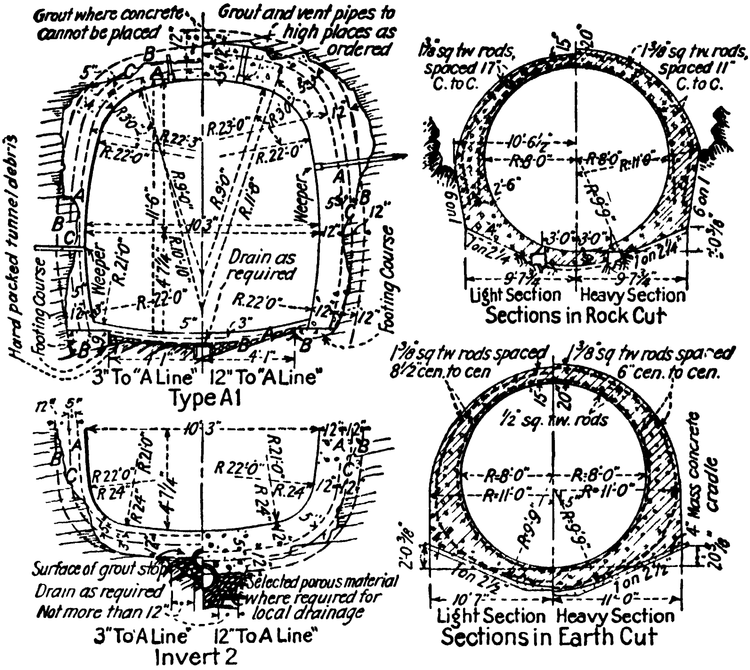
Bench Flume

FIG. 87. Rectangular reinforced-concrete flumes.

180. Pipe Lines. When rough topography or other conditions make the use of a canal impracticable, a pipe line with water flowing under pressure in it may be used. Since pipe lines can, in general, follow shorter routes than canals, they may be cheaper to construct. A pipe line crossing a valley and lying on or under the ground surface with each end of the pipe line in or near the hydraulic grade line is called a siphon, although the pressures in it are normally greater than atmospheric. A true siphon is that part of a pipe line in which the water flows above the hydraulic grade line. True siphons should be avoided because of difficulties to be expected from air leaks and from gases coming out of solution and accumulating at the top of the siphon with resulting reduction in siphon capacity. A force main is that portion of a pipe line through which water is pumped, usually to a higher elevation. Booster pumps may be used to increase the pressure in long pipe lines. The discharge pipe from a booster pump would not, necessarily, be classified as a force main, unless the water were lifted to a higher elevation.

Materials used for pipes in aqueducts include concrete, cast iron, steel, wood, and asbestos cement. The material selected depends upon such conditions as (1) assurance that operation will not be interrupted; (2) annual cost; (3) suitability for conditions which include loads, pressures, and the corrosive characteristics of the water and the soil in contact with the pipe; (4) accessibility of the site and ease of transportation of materials; and (5) availability of skilled labor for construction.

The annual cost depends on the first cost, operating and maintenance cost, and depreciation. Many factors such as length of life, painting, interior roughness, facilities for repair, and location must be considered in connection with determinations of annual cost. Each particular case must be considered separately, as there is no general rule that can be applied.



a b

FIG. 88. Aqueduct sections: *a*, sections of Shandaken Tunnel; *b*, sections of Croton Aqueduct. (*From Report of New York Board of Water Supply.*)

Adaptability of the material for the service to be performed requires consideration of the quality of water and soil and the internal pressure to be carried. Under some conditions any one of the materials might be used successfully, whereas for others some of them would be quite unsuitable.

Likewise for some installations, transportation and construction facilities might be equally suitable for all the materials, but where the line is located in a region of difficult access, the use of materials that can be easily transported in small pieces of light weight and assembled at the site is mandatory. In some localities skilled labor and equipment for construction operations such as riveting or welding are not readily available. Deference to local manufacturing industries may also be a factor in the selection of pipe material. Characteristics of materials are discussed in Chap. XVIII; stresses are discussed in Chap. X.

Circular cross sections are most commonly used for pipe lines under high pressure, since the stresses can be most economically carried under tension. Typical cross sections of concrete pipes are illustrated in Figs. 157, 158, and 159. However, large cross sections, either with or without cover are constructed of other shapes, such as those shown in Fig. 88. Such shapes are better suited to resisting heavy external loads and light internal pressures and are easier to construct than a circular shape.

To assure safety, permit inspection, and facilitate operation and maintenance, pipe lines should be provided with gate valves, check valves, drainage valves, air valves, and manholes. Surge tanks or other surge-control equipment may be necessary on some lines. Gate valves may be placed about 1,000 to 1,500 ft. apart, near manholes, with a drainage valve at a low point between the gate valves to permit emptying the pipe between valves for inspection and repair. A check valve should be placed at the upstream side of the beginning of each rise in the pipe line to prevent reversal of flow in an emergency. A gate valve may be placed near to and on a convenient side of the check valve to permit inspection and repair. The check valve should be designed to close slowly enough to avoid dangerous water hammer. Drainage valves should be placed at all low points to permit removal of deposited silt and the emptying of the pipe. An air valve should be located at each high point to allow the escape of air and gases and to admit air sufficiently rapidly to prevent the creation of a partial vacuum in the pipe line.

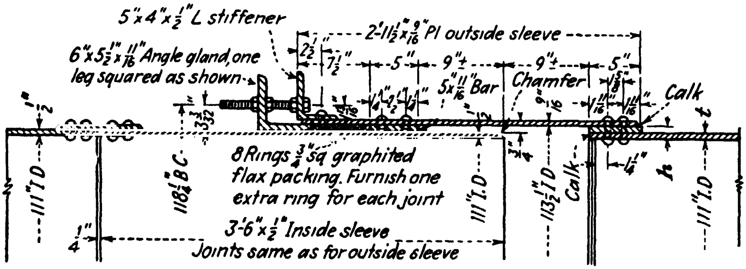

5"×4"×½" L stiffener
6"×5½"×11/16" Angle gland, one leg squared as shown
2'-11½"×9/16" Pl outside sleeve
2½"
7½"
5"
9"±
9"±
5"
5×11/16" Bar
Chamfer
5/8"
Calk
½"
3 3/32"
¼"
2½"
¼"
9/16"
11/16"
11/16"
t
8 Rings ¾" sq graphited flax packing. Furnish one extra ring for each joint
111" I.D
118¼" B.C.
111" I.D
113½" I.D
Calk
1¼"
111" I.D
¼"
¾"
3'-6"×½" Inside sleeve
Joints same as for outside sleeve

FIG. 89.—Detail of a steel-pipe expansion joint.

Expansion joints may be required on long pipe lines, particularly on steel pipe. Figure 89 shows a type of joint used by the U.S. Bureau of Reclamation. When several expansion joints are used in a long line, it is necessary to construct anchors between them in order to force the movement into the joint designed to take it. Where piers are used to support the full weight of steel pipe, they should be spaced 20 to 40 ft. apart and be constructed so that one-sixth to one-fourth, or more, of the pipe circumference will bear upon them. They should be constructed to resist the overturning forces resulting from friction due to longitudinal movements of the pipe. In general, a friction coefficient of not less than 0.5 between pipe and pier should be used in design.

181. Diameter of Pipe through Which Water Is Pumped.[1] The diameter of a pipe through which water is pumped should be such as to give the lowest cost of the items of first cost, maintenance, including pumping, and renewal. The diameter of this lowest cost or economical pipe can be determined by the following procedure:

1. Formulate an expression for the capitalized cost, or total annual cost of the pipe line, assuming any convenient terms.
2. Express all the variables in the preceding expression in terms of one variable
3. Differentiate with respect to that variable, equate to zero, and solve. The result will be the economical value of the variable.

The above procedure will be applied to the determination of the economical diameter of a long cast-iron pipe line through which water is being pumped. The following nomenclature will be used:

a = cost of iron, cents per lb.
$B = 0.00159L + 0.0006Y + (0.055 + 0.0145D)W$
C = coefficient in Hazen and Williams formula, taken as 100
d = diameter of pipe, in.
D = depth of trench, ft.
L = cost of lead, cents per lb.
p = cost, in cents, of pumping 1 million gal. 1 ft. high
Q = rate of flow through pipe line, c.f.s.
r = annual rate of interest, plus sinking fund rate of depreciation, plus rate of other annual charges
s = slope of hydraulic grade line
V = velocity, f.p.s.
W = wage rate for common labor, cents per hr.
Y = cost of yarn (oakum), cents per lb.

The annual cost, per foot of length, of a cast-iron pipe was shown by Maury,[2] including first cost and renewal and laying in the trench, to be expressable in the form

$$\text{Cost} = Bdr + 2ard^{1.5} \tag{1}$$

The cost of pumping through the pipe line can be expressed as

$$\text{Pumping} = 236pQs \tag{2}$$

---

[1] See also SARCHET, B. R., and A. P. COLBURN, "Economic Pipe Size," *Ind. Eng. Chem.*, September, 1940, p. 1240.

[2] MAURY, D. H., "Diagram Showing the Cost of Laying Cast-iron Pipe," *Eng. News-Record*, Vol. 88, p. 779, 1922.

The total annual cost for all charges is, therefore,

$$\text{Total cost} = Bdr + 2ard^{1.5} + 236pQs \qquad (3)$$

In taking the second step of the outlined procedure it is necessary to express the two variables d and s in terms of each other. This can be done by rearranging Hazen and Williams formula into

$$s = 165\,\frac{Q^{1.85}}{d^{4.86}} \qquad (4)$$

Substituting this value of s in expression (3) there is obtained

$$\text{Total cost} = Bdr + 2ard^{1.5} + 39{,}000p\,\frac{Q^{2.85}}{d^{4.86}} \qquad (5)$$

Differentiating with respect to d and equating to zero

$$Brd^{5.86} + 3ard^{6.36} = 189{,}000pQ^{2.85} \qquad (6)$$

which can be expressed approximately as

$$d = 7.25\left(\frac{p}{r(B + 3a)}\right)^{0.167} Q^{0.176} \qquad (7)$$

The economical velocity can be determined from a combination of

Table 43.—Economical Velocities for Different Diameters and Rates of Flow in Cast-iron Pipe[1]

Diameter, inches	Values of constants[2]			Quantity, second-feet	Values of constants[2]		
	$L = 10$ $Y = 15$ $a = 4$ $r = 0.07$ $p = 4$ $W = 100$ $D = 6$ $B = 14.2$ $k = 0.441$	$L = 5$ $Y = 10$ $a = 3$ $r = 0.06$ $p = 6$ $W = 50$ $D = 4$ $B = 5.66$ $k = 0.1466$	$L = 3$ $Y = 4$ $a = 1.5$ $r = 0.04$ $p = 15$ $W = 30$ $D = 2$ $B = 2.33$ $k = 0.0182$		$L = 10$ $Y = 15$ $a = 4$ $r = 0.07$ $p = 4$ $W = 100$ $D = 6$ $B = 14.2$ $k = 0.441$	$L = 5$ $Y = 10$ $a = 3$ $r = 0.06$ $p = 6$ $W = 50$ $D = 4$ $B = 5.66$ $k = 0.1466$	$L = 3$ $Y = 4$ $a = 1.5$ $r = 0.04$ $p = 15$ $W = 30$ $D = 2$ $B = 2.33$ $k = 0.0182$
4	2.97	1.99	0.93	0.1	2.75	1.90	0.95
6	3.18	2.12	1.00	0.5	3.14	2.16	1.08
8	3.36	2.25	1.05	1.0	3.31	2.29	1.14
10	3.50	2.34	1.10	5	3.78	2.61	1.30
12	3.61	2.42	1.13	10	4.00	2.77	1.38
16	3.82	2.56	1.20	25	4.33	3.00	1.49
18	3.91	2.62	1.23	50	4.57	3.16	1.57
24	4.12	2.76	1.29	100	4.86	3.37	1.68
36	4.42	2.96	1.39				

[1] Velocities are given in feet per second.

[2] For nomenclature, see p. 171.

$$k = \frac{r(B + 3a)}{p}$$

expression (7) and $Q = AV$, so that

$$V = 3.5Q^{0.05}\left(\frac{r(B + 3a)}{p}\right)^{0.33} \tag{8}$$

$$V = 3d^{0.1}\left(\frac{r(B + 3a)}{p}\right)^{0.35} \tag{9}$$

Economical values of the velocity of flow[1] are entered in Table 43. They have been computed from an approximate formula based on Flamant's formula for flow. It is expressed as

$$V = 3.1d^{3/11}\left(\frac{r(B + 3a)}{p}\right)^{4/11} \tag{10}$$

182. Diameter of a Pipe through Which Water Flows by Gravity. To secure the greatest economy, the diameter of a single pipe through which water flows by gravity should be such that all the head available for causing flow is consumed by friction. The economical diameter is, therefore, that diameter which is found by substituting this head and the required rate of flow in any of the formulas for the flow of water in pipe given in Sec. 40.

183. Tunnels. Tunnels are constructed in aqueducts to pass through mountains and cross under rivers and in situations in which surface lines or open-cut excavation would be impracticable. When a tunnel and a surface location are equally feasible, the tunnel will ordinarily involve a greater first cost, but the capitalized cost of the two may show the tunnel to be cheaper. The decision between them may lie in the availability of funds or in the consideration of safety and uninterrupted service.

In a grade tunnel the hydraulic grade line coincides with the surface of the flowing water. In a pressure tunnel the hydraulic grade line is above the tunnel. Both types of tunnel are in use, and both types may resist external forces due to surrounding ground pressures and ground-water pressures. Some tunnel sections designed to resist external forces are shown in Fig. 88.

Tunnels may or may not be lined, depending on the materials through which they are driven, the amount of head available, ground-water conditions, and other considerations. In earth and in crumbling rock, lining is essential. A reinforced concrete or metal lining must be provided in pressure tunnels at places where the weight of the earth or rock cover is not sufficient to resist safely the bursting pressure within the tunnel. The hydraulic properties of a tunnel are greatly improved by lining; hence the loss of head due to friction for a given capacity is much smaller in a lined tunnel of a given cross-sectional area than in an unlined tunnel of the same area. The entrance of objectional ground water into a tunnel or the escape of water from it may be prevented by a watertight lining of brick, concrete, steel, cast iron, or concrete-protected metal or by grout, placed under pressure, back of the lining. When the tunnel is driven through material not subject to caving or disintegration, and other considerations are not of sufficient weight to warrant the expense, the lining is frequently omitted.

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[1] See also CAMP, T. R., "Economic Size for Water Distribution Systems," *Trans. A.S.C.E.*, Vol. 104, p. 190, 1939.

184. Cross Sections of Aqueducts. The cross-sectional area of the waterway of an aqueduct is determined by the head available, limiting velocities, and other conditions. When adequate head is available, the area of waterway is determined by the maximum allowable velocity. Table 44 gives maximum velocities recommended for various aqueduct materials and types of structures. The minimum velocity should be sufficient to prevent the deposition of sediment in the channel.

TABLE 44.—MAXIMUM VELOCITIES IN AQUEDUCTS

Type of structure	Maximum average velocity, feet per second
Cut-and-cover sections	
Concrete lined	15.0
Brick lined	18.0
Pipes:	
Steel and cast iron	12.0–20.0
Concrete	10.0–15.0
Wood	15.0
Tunnels	
Unlined	12.0
Concrete lined	10–15
Steel lined	12–20
Flumes—all types	Less than critical velocity
Canals:	
Earth:	
Ordinary	2.5– 3.0
Sand .	1.0– 2.0
Hard gravel or firm clay	5.0– 6.0
Rock	8.0–15.0
Concrete lined .	10.0–15.0

The sizes and shapes of cross sections in a long aqueduct are usually varied to meet conditions of external loads and to provide the greatest economy. The shape should be designed to support the external loads, and the area of waterway should give the least cost and consume the available head in the most economical manner. For example, where a fixed amount of head loss is available between the upper and lower ends of an aqueduct, part of which is to pass through a hard rock tunnel and the other to be constructed as a canal, it may be more economical to consume most of the available head in the tunnel, to permit the smallest possible bore, than to divide the head loss between the two sections proportional to their length. The most economical distribution of head losses in an aqueduct passing through different materials, if the total head loss is fixed, can be determined by the following procedure:

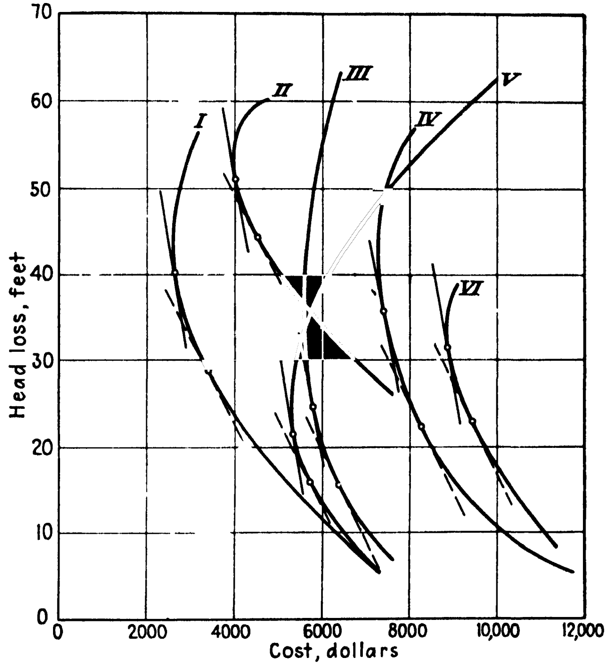

FIG. 90.—Diagram of parallel-tangent method for determining the minimum cost of an aqueduct.

1. Construct a set of curves whose abscissas are the total cost of the full length of each section of the aqueduct for the shape of cross section desired for that part of the aqueduct and whose ordinates are the head lost in that part of the aqueduct. A typical set of cost-head loss curves is shown in Fig. 90. Data for such curves must be obtained by designing various conduit shapes suitable for the materials through which each is passing and estimating the total cost of construction. The head loss is determined through the application of a satisfactory formula of flow.

2. By trial and error construct a series of parallel lines, each tangent to one curve, in a position such that the sum of the ordinates to all the points of tangency is equal to the total available head loss. Then the sum of the abscissas to the points of tangency, *i.e.*, the total cost of the aqueduct, is a minimum. The aqueduct can be constructed for the least cost by adopting the head losses, shapes, and sizes in each section, as indicated in this study.

In the example illustrated in Fig. 90 the minimum cost for the six sections, for a total head loss of 147 ft., is $37,500. The head loss for each section can be read from the figure. For example, the head loss to be allowed in Section I is 28 ft. The shape and area of cross section that will give this head should be constructed.

185. Surge. Surge in closed aqueducts is similar to the phenomenon of water hammer discussed in Sec. 192, and in open channels it is similar to the traveling wave, or bore. Both result from sudden changes in the velocity of flow through the conduit. Water hammer may create collapsing or bursting pressures in closed conduits or overflows in open channels. They may be controlled in closed conduits by the methods discussed in Sec. 193 and in open conduits by proper design under conditions of unsteady flow.[1]

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[1] See also "Delaware Aqueduct-V. Controlling the Flow," *Eng. News-Record*, July 18, 1940, p. 73.

CHAPTER X

STRESSES IN PIPE

186. Loads and Stresses. Conduits carrying water under pressure develop stresses due to the following loads and conditions:

1. Internal bursting pressure produces circumferential tension and longitudinal tension at bends and dead ends.
2. Temperature changes produce longitudinal tension and compression.
3. Flow around bends produces longitudinal tension.
4. Weight of pipe, water, backfill, and other superimposed loads, and atmospheric pressure and reactions at foundations or other supports produce flexural tension and compression.

Shear may result under the application of any of the loads, depending on the manner of the load application and the construction of the conduit.

187. Internal Bursting Pressure. Internal pressures are caused by static pressure and water hammer. The tension T in a unit length of pipe, having a radius R and operating under an intensity of pressure p, is expressed by the formula

$$T = pR \tag{1}$$

The value of p for pressures due to static head is determined by the relation $p = wh$, in which w is the weight per unit volume of water and h is the maximum possible head to which the pipe may be subjected. The static head is measured from the highest possible position of the hydraulic grade line to a point in the pipe. If water hammer can occur, the head to which the pipe may be subjected is the sum of the static head and the head developed by the pressure rise due to water hammer.

188. Temperature Stresses. Longitudinal stresses due to changes in temperature can be computed from the expression

$$S = ETC \tag{2}$$

where S = intensity of stress due to change in temperature
E = modulus of elasticity of the metal
T = change in temperature
C = coefficient of expansion of the metal

If the pipe is to be allowed to expand and contract with changes in temperature, expansion joints must be provided. The movement of the pipe can be computed from the expression

$$M = LCT \tag{3}$$

where M = change in length of the pipe
L = length of the pipe affected
C = coefficient of expansion of the metal
T = change in temperature

Temperature stresses and changes in length are usually not considered in the design of cast-iron distribution pipes underground. In the design of long steel conduits and of pipes conveying hot liquids, allowance should be made for temperature stresses unless adequate expansion joints are provided to reduce the effects of temperature change.

189. Flow around Bends. Longitudinal tension resulting from the flow of water around a bend in a closed pipe can be completed from the expression

$$T = \frac{WAV^2}{g} + pA \tag{4}$$

where W = unit weight of water
A = cross-sectional area of the pipe
V = velocity of the flow of water
g = acceleration due to gravity
p = intensity of internal bursting pressure

The first term in formula (4) represents the stresses due to the flow of water around the bend. The load is produced by the centrifugal force resulting from the mass of water moving around the arc of the bend.

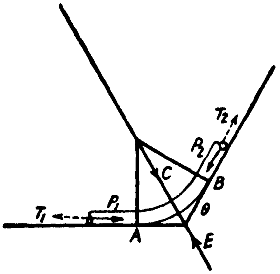

FIG. 91.—Diagram of stresses in a bend.

The force T may be carried as longitudinal tension in the shell and joints of the pipe, or it may be supported by a buttress placed at the bend. If a buttress is placed along the line of action of the resultant, *e.g.*, at E in Fig. 91, the buttress must resist the resultant force whose magnitude is

$$E = 2A\left(\frac{WV^2}{g} + p\right)\sin\frac{\theta}{2} \tag{5}$$

Bends and tees that are not supported by a buttress must be adequately anchored. Anchors may consist of straps, tie rods, bolted flanges, or a combination of them.

190. Flexural Stresses. Flexural stresses result from the weight of the pipe, the weight of the water, from superimposed loads that tend to change the shape of the pipe, and from longitudinal loads that are resisted by the pipe acting as a beam, a column, or a tie rod. Combinations of transverse and longitudinal loads may result in combinations of arch, beam, column, and tie-rod actions in the pipe. Such loadings and conditions produce secondary flexural stresses resulting in transverse tension, compression, and shear that are difficult to compute because their magnitudes depend, to some extent, on the local deflections or strains produced in the pipe. Their computation must be based on the application of the principles of the theory of elasticity to the structural material of the conduit. Simple formulas are not available. When the external loads are known, the internal stresses and strains may be computed by reference to texts on structural analysis.

191. External Loads on Buried Pipes. Loads on buried pipe are formulated by Anson Marston[1] and are discussed by Spangler.[2] Marston's formulas are expressed as follows:

For backfill load on ditch conduits

$$W = C_d w B^2 \tag{6}$$

For superimposed, dead loads along trench

$$W = C_{us} B U_{us} \tag{7}$$

For moving, short loads, across trench

$$W = \frac{I_c C_t T}{3} \tag{8}$$

where B = width of trench, slightly below the top of the pipe, ft
C_d, C_{us}, and C_t = coefficients depending on the conditions surrounding pipe in trench
I_c = impact coefficient, lying between 1.5 and 2.0, depending on the speed of the moving load
T = concentrated superimposed load, lb.
W = load on pipe per foot of length of trench, lb.
w = unit weight of backfill material, lb. per cu. ft.

A few representative values of the coefficients are shown in Table 45.

192. Water Hammer. Water hammer is a pulsation of pressures above and below the operating pressure, resulting from rapid deceleration or acceleration of the velocity of flow of water in a closed conduit. The terms surge, ram, and water hammer are applied synonymously to this phenomenon. Maximum pressure changes usually occur when the water is brought quickly to a stop. The maximum pressure produced can be formulated as

$$P = \frac{4{,}665Vw}{144g} \sqrt{\frac{1}{1 + (Kd/Et)}} \tag{9}$$

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[1] *Iowa Eng. Expt. Sta.*, *Bull* 31, 47, 57, 76, 96, 108, and 112.

[2] SPANGLER, M. G., "Underground Conduits—An Appraisal of Modern Research," *Proc. A.S.C.E.*, June, 1947, p. 855.

where V = velocity of flow of liquid, f.p.s.
w = weight per unit volume of liquid
E = modulus of elasticity of the pipe material
K = bulk modulus of elasticity of the fluid (294,000 p.s.i. for water)
d = diameter of pipe, in.
t = thickness of pipe shell, in.
g = acceleration due to gravity, 32.2 ft. per sec.^{2}

The velocity V_w at which the pressure wave travels through the pipe is

$$V_w = 4{,}665\sqrt{\frac{1}{1 + (Kd/Et)}} \qquad (10)$$

Values of K/E for water and different pipe materials are given in Table 46.

If L is the length of pipe from gate to point of relief, the time of closure must be less than or equal to $2L/V_w$ to develop maximum pressure

TABLE 45.—SOME VALUES OF COEFFICIENTS IN MARSTON'S FORMULAS

Ratio H/B*	Damp top soil and dry and wet sand		Saturated yellow clay		Depth of cover	Conduit breadth, ft				
						1	2	3	5	10
	C_d	C_{us}	C_d	C_{us}		Values of C_t				
1 0	0 85	0 72	0 90	0 80	1	0 52	0 78	0 86	0 90	0 92
2 5	1.70	0 14	1 91	0 57	3	0 125	0 25	0 34	0 44	0 53
5 0	2 15	0 19	3 02	0 33	5	0 045	0 10	0 15	0 23	0 31
10	2 92	0 04	4 01	0.11	7	0 02	0 05	0 08	0 12	0 20
15	3 01		4 34	..	10	...	0.02	0.04	0 06	0 10
Inf	3 03		4 50							

* H is depth of cover over top of pipe, in feet.

An approximate value of the pressure rise that occurs when the actual time of closure T_a is greater than the critical time $T_c = 2L/V_w$ (greatest time that will produce maximum pressure rise) may be obtained by multiplying the maximum pressure rise from formula (9) by the ratio T_c/T_a.

TABLE 46.—RATIOS OF K/E*

Material	K/E
Plate steel.	0 01
Cast iron	0 02-0 022
Concrete	0 10
Wood	0 20

* ANGUS, R. W., "Water Hammer Pressures in Compound and Branched Pipes," *Trans A S C E.*, October, 1939, p. 340

The foregoing theory gives approximate values for single lines of pipe uncomplicated by interconnections with networks, branch pipes, dead ends, or multiple sources of input such as a combination of a pump and storage reservoir. A graphical solution for complex problems of water hammer has been developed by Robert W. Angus.[1]

193. Causes and Prevention of Water Hammer.[2] Water hammer is frequently encountered in waterworks practice. It may be caused by quick opening or closing of valves or gates; the sudden starting, stopping, or variation in speed of pumps; breakage of pipe lines; and other conditions.

Some methods and devices used for the prevention or control of water hammer[3] include (1) relief valves, (2) air valves, (3) air chambers, (4) surge tanks or towers, (5) surge suppressors, and (6) slowly closing valves. If the cause of the water hammer is at the end of the pipe toward which water is flowing, relief devices must be set to minimize an increase in pressure; if at the other end of the pipe line, they must minimize a decrease in pressure.

Relief valves may be actuated by a slight change in pressure, of as little as 10 p.s.i., to open wide in a few seconds to discharge water to waste if an increased pressure is to be relieved or to admit water or air if a decrease of pressure is to be relieved. Air-inlet valves are designed to open at a slight pressure below normal operating pressure to admit air to avoid the creation of a partial vacuum. Closed air chambers may be used to relieve either a high pressure or a low pressure.[4] They are particularly useful near a valve or faucet on plumbing. Since closed air chambers must be filled with air at the time they are called on to function, the chamber should be set on a high point on the pipe line to permit the accumulation of air in it.

A surge tower, in the form of an open tank "floating on the line" may be connected to a pipe line to take water from it or to deliver water to it as the conditions demand. The design of the capacity of such tanks is discussed in Sec. 194.

Surge suppressors are mechanical devices in which the energy of the surge is absorbed in springs, compressible fluid, or other mediums, to be slowly returned to the water as the suppressor recovers from the shock of the ram. They are applicable particularly to small pipes and in plumbing, rather than to long pipe lines in aqueducts.[1] The term suppressor is applied to other devices for the control of water hammer.

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[1] *Trans. A.S.C.E.*, October, 1939, p 340.

[2] See also LAPWORTH, C. F , "Surge Control in Pipe Lines," *Trans. Inst. Water Eng.*, Vol. 49, p. 29, 1944; and *Surveyor*, July 7, 1944, p. 321.

[3] See also BENNETT, R., "Water Hammer Correctives," *Water & Sewage Works*, April, 1946, p R-106.

[4] See also LIBBY, M A., "Pneumatic Surge Tank Solves Water Hammer Problem," *Water & Sewage Works*, March, 1946, p. 98.

Water hammer resulting from the sudden cutting off of power from a pump may be diminished by increasing the inertia of the moving parts of the pump so that, acting as a flywheel, the pump is brought more slowly to a stop.[2] The use of check valves is discussed in Sec. 317. One type of valve designed to prevent water hammer is kept open by the velocity of the flowing water. When the velocity slows down, the valve closes before the direction of flow is reversed to build up water pressure.

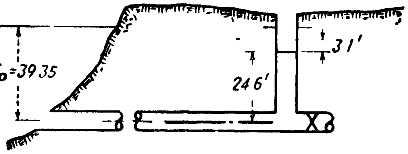

FIG. 92.—Surge tower on intake line.

It is seldom that any two cases of water hammer are exactly alike. Each must be studied, and one or more of the devices or methods available for its suppression must be used alone or in combination with others.

194. Surge-tower Design. A method for the determination of the capacity of a surge tower and the rise and fall of water in it has been suggested by Halmos.[3] The following is an example of its application:

Example: Let it be desired to determine the rise of water level in the surge tower shown in Fig. 92 when the pump is suddenly stopped.

Data: Length of conduit = 4,920 ft; velocity of flow in pipe line = 10.17 f.p.s.; velocity of pressure wave in conduit, and in wet well, computed from formula (10) in Sec. 192, = 2,950 f.p.s.; $A_a = A$.

Nomenclature:

A = cross-sectional area
A_a = cross-sectional area of tunnel
A_t = cross-sectional area of tower
a = velocity of pressure wave
C' = coefficient of discharge of opening into surge tower
g = acceleration due to gravity
H = head
H_b = head at base of surge tower
H_0 = static head

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[1] See also WHITE, J. M., "Application of Surge Suppressor in Water Systems," *Water Works Eng.*, Mar. 25, 1942, p. 304.

[2] See also LAPWORTH, C. F., "Control of Surge in Pipe Lines," *Jour. A.W.W.A.*, October, 1945, p. 1036.

[3] HALMOS, E. E., "The Effect of Surge Tanks on the Magnitude of Water Hammer in Pipe Lines," *Symposium on Water Hammer*, p. 72, published jointly by American Society of Civil Engineers and American Society of Mechanical Engineers 1933.

L = length of pipe line
T = time valve closes or pump stops
V = linear velocity of water
μ_a = period of conduit = $2L/a$ = period of surge tower

Solution: 1. Determine H_b, where A_b = zero. The highest possible pressure, without a surge tower, is equal to $2K_aH_o$ where

$$K_a = aV/2gH_o = 2{,}950 \times 10.17/64.4 \times 39.35 = 11.85$$

Hence

$$H_b = 2 \times 11.85 \times 39.35 = 933.5 \text{ ft.}$$

2. Compute H_b where $A_a = A$

Under these conditions

$$H_b = \frac{(2K_aH_o)(A_b)}{(A_a + A_b)}$$

Hence

$$H_b = \frac{933.5 \times 1}{2} = 466.7 \text{ ft.}$$

Because of the time required to close the valve, and because of the cushioning effect of the surge tower, the pressure rise will be less than that shown above. Under this condition

$$H_b = \frac{(K_a)(H_o)(A_b)(\mu_b)}{(T)(A_a)}$$

In this case

$$\mu_b = 2 \times 24.6/2{,}950 = 0.0167$$

Then

$$H_b = 11.85 \times 39.35 \times 1 \times 0.0167/2.5 \times 1 = 3.12 \text{ ft.}$$

where T = 2.5 sec.

Enger[1] states that

$$H = \frac{a}{g}V - B_r^2\left(\frac{a}{g}\right)^2\left[\sqrt{\left(\frac{H_og^2}{B_r^2a^2} + \frac{Vg}{B_r^2a} + \frac{1}{4}\right)} - \frac{1}{2}\right] \tag{11}$$

in which

$$B_r = \frac{CA_b}{A_a}\sqrt{2g}$$

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[1] ENGER, M. L., *Symposium on Water Hammer*, p. 97, published jointly by American Society of Civil Engineers and American Society of Mechanical Engineers, 1933.

CHAPTER XI

IMPOUNDING RESERVOIRS

195. Function. An impounding reservoir is a basin constructed in the valley of a stream to store water during excess stream flow and to supply water when the flow of the stream is insufficient to meet the demand for water.

196. Site Characteristics. Certain site characteristics are desirable for an impounding reservoir. These, with the favorable features of each, are:

1. Geology:
   - *a.* Surface or immediately underlying soil or rock of such composition and structure that it will offer high resistance to the percolation of water through it.
   - *b.* The absence of objectionable soluble minerals or salts which would affect the quality of the stored water.
2. Topography:
   - *a.* A narrow opening to reduce the length of the dam.
   - *b.* A rapid widening of the valley or canyon above the dam site to afford greater average volume per foot of height and length of dam.
   - *c.* Steep side slopes to reduce surface area per unit of volume.
3. Real estate:
   - *a.* Inexpensive land to be submerged.
   - *b.* Absence of railways, highways, and water-power developments which might be submerged or destroyed.
4. Cultural:
   - *a.* Freedom from trees and underbrush which may have to be removed.
   - *b.* Absence of objectionable vegetation and marshy land which might impart objectionable color, taste, or odor to the stored water.

The selection of a site for an impounding reservoir is predicated upon the fact that the yield of the stream will serve the intended purpose.

Reservoir sites possessing all these characteristics to a desirable degree are rarely found.

It is possible in many instances to remedy the defects of a dam and reservoir site by artificial means. In fact, there are cases in which it is absolutely necessary to do so in order to make use of a site. For example, at the site of Echo Reservoir, constructed by the Bureau of Reclamation on the Weber River in Utah, an exceptionally difficult situation was remedied by the use of cutoff walls and clay puddle extending through the pervious upper layers to the underlying impervious clay. In addition, a clay blanket extending a considerable distance above the upstream toe of the dam was spread over the stream bed. Pressure grouting may be required when the foundation material is composed of porous or shattered rock.

The sanitary features of reservoir sites, their maintenance, and their use as recreational centers are discussed in Sec. 448.

197. Preparation of Site.[1] The storage of water in contact with vegetation or decaying organic matter may be undesirable. Vegetation is removed from reservoir sites by cutting down and removing trees and undergrowth and by burning off grass and weeds. Swampy areas are sometimes drained, scooped out, or filled in. In scooping them out, depressions in which organisms may grow should not be left in the reservoir. It may be considered necessary to remove highly organic surface soil such as peat, cultivated land, or boggy muck. Some authorities consider it desirable to strip all reservoir sites in this thorough manner, but the expense is usually a deterrent to the practice. Shallow areas should be avoided, as they are conducive to the growth of weeds and aquatic plants.

Experience shows[2] that the stripping of a reservoir site is an expedient which will obviate difficulties when the reservoir is first thrown into service, but in time there is little difference between a stripped and an unstripped reservoir. In general, the stripping of reservoir sites is not to be recommended because of the expense, which is not commensurate with the results. Trees, shrubs, and undergrowth which may protrude above the surface of the water at any stage should be removed. Human habitations, stables, barns, and other possible sources of pollution should be removed from the site.

198. Requisite Capacity. The basic information needed to determine the required capacity of an impounding reservoir is a hydrograph of the flow of the stream for the longest and driest period known or predicted and data on the probable demands for water during that period. With this information available the first step in the determination of the requisite capacity of an impounding reservoir is to construct a mass diagram of the flow of the stream. This is a curve whose abscissas are time and whose ordinates are total flow from the start of observations to the time observed. The information may be taken from a hydrograph, as shown in Fig. 93.

In the construction of the mass diagram, the length of any ordinate, such as *ab*, or *cd*, is proportional to the area under the hydrograph and to the left of the ordinate extended. If the rate of demand is constant so that it can be represented by a horizontal line *ef* on the hydrograph, the mass diagram can be drawn as a straight line *og*.

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[1] HAZEN and WHIPPLE, "Stripping Water Works Reservoirs," *Eng. News*, Vol. 73, p. 858, 1915.

[2] *Jour. A.W.W.A.*, 1908, p. 195.

Now draw lines parallel to *og* and tangent to the mass diagram of supply, as *hj*, *kl*, *mn*, and *pq*. The greatest vertical distance between tangents in a pair, when measured to the scale of the ordinates of the mass diagram, represents the requisite capacity of the impounding reservoir.

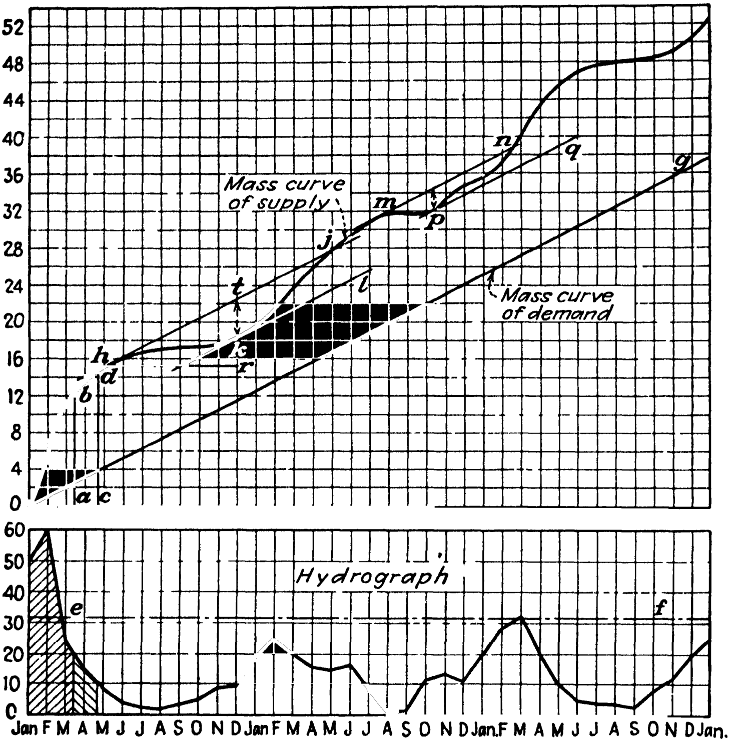

FIG. 93.—Hydrograph and mass diagram for impounding reservoir.

That the vertical distance between tangents in any pair represents the deficit of supply over demand for the particular deficit represented can be proved as follows: At the point *h*, draw a horizontal line *hr*. At the point of tangency *k*, erect a vertical line *rkt*. The scalar distance *rt* represents the accumulated demand from the point *h*. The scalar distance *rk* represents the accumulated supply from point *h*. The scalar distance *kt*, therefore, represents the accumulated deficit. Any other accumulated deficit from *h* to *j* will be less than *kt*. Hence *kt* represents the requisite capacity of the impounding reservoir to supply the deficit during the period from *h* to *j*.

Other information supplied by the mass diagram includes (1) the dates that the reservoir is full, points *h* and *m*; (2) the dates that the reservoir is empty, points *k* and *p*; (3) if the flow of the stream is insufficient to fill the reservoir, tangents to low points, such as *kl* and *pq*, if extended to the left will not intersect the mass diagram; and (4) if tangents, such as *hj* and *mn*, extended to the right do not intersect the mass diagram, no reservoir can be constructed large enough to supply the deficit in the flow of the stream.

In the event that the rate of demand is not constant, the procedure should be to draw both the mass diagram of demand and the mass diagram of supply. Then construct a third curve whose abscissas are time and whose ordinates represent the difference between the two mass diagrams. If horizontal lines are drawn tangent to this third curve, the scalar distances between tangents in a pair of tangents will have significance corresponding to the scalar distances between the sloping tangents where the rate of demand is constant. Other information can be obtained from this curve similar to that obtainable from the sloping tangents.

199. Losses from Reservoirs. Evaporation, seepage, and compensation water represent losses from the stream and reservoir that may be considered as negative increments of stream flow. Compensation water is water that must be passed downstream from the dam to compensate water rights of persons below the reservoir. These increments may, at times, be greater than the flow of the stream so that it is possible for a mass diagram of supply, with allowance for losses, to have a negative slope. Another loss of capacity experienced by impounding reservoirs results from the partial filling of the reservoir by silt. This loss does not appear on the mass diagrams.

200. Measurement of Capacity. The volume of water that can be held in an impounding reservoir may be approximately determined by multiplying average surface areas between contours by the contour interval, or the prismoidal formula may be used, in which

$$V = \frac{h}{6}(A_1 + 4A_2 + A_3)$$

where V = volume between contours, corresponding to surface areas A_1 and A_3

A_1, A_2, A_3 = respective areas enclosed within a lower, a median, and an upper contour, whose contour interval is h

h = contour interval

The summation of separate volumes determines the total volume. The capacity of the reservoir is usually expressed in gallons, cubic feet, or acre-feet, depending on local usage.

If the depth of the water in the reservoir increases, the capacity of the reservoir and the surface area of the water in it will increase. Curves showing the relation between depth and capacity and depth and area of water surface are known, respectively, as capacity and as area curves. Such curves for the Geist reservoir at Indianapolis are shown in Fig. 94.

When the requisite capacity of a reservoir has been determined, as explained in Sec. 198, its depth to give this capacity can be read from a figure such as Fig. 94.

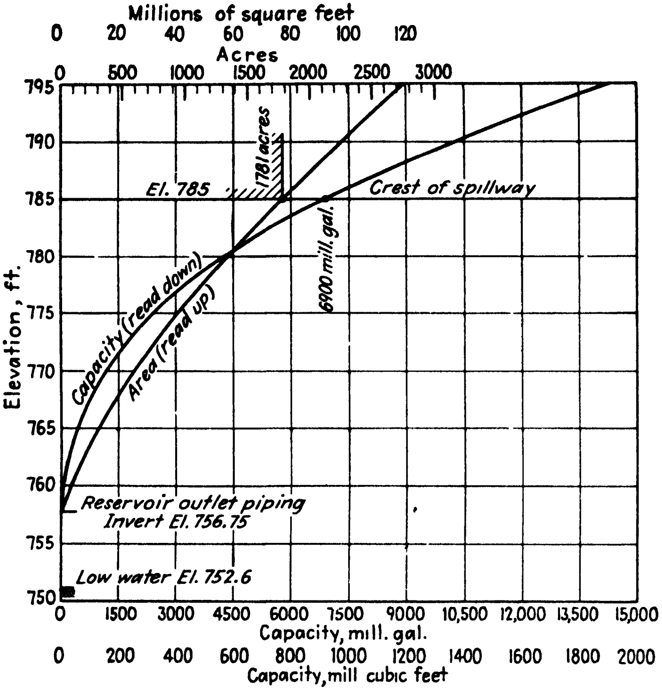

FIG. 94. Area and capacity curves, Geist Reservoir. (*From Jour. A.W.W.A.*, *Decemb* 1945, *p.* 1325.)

201. Silting of Reservoirs. Loss of capacity of an impounding reservoir due to the deposition of silt in it may impair, if not destroy, the usefulness of the reservoir in a few years. Annual losses up to 6 per cent of capacity are reported by Brown.[1] Witzig[2] has formulated the silt-carrying capacity of streams as $G_s = C_sQ^x$, in which G_s is the suspended sediment carried in the stream, in tons per second; C_s is a coefficient; Q is the discharge of stream in cubic feet per second; and x is an exponent. Although quite inaccurate the formula may be useful in estimating the quantities of silt delivered to a reservoir, where values of C_s and of x have been determined by observation. Witzig reports values of C_s of the order of 9.5×10^{-9} to 2.55×10^{-12} and values of x from 1.78 to 2.77. The average annual suspended silt load can be computed by a combination of the "silt rating curve," as expressed by the above formula, and the discharge-frequency curve. Silt-bearing water does not always discharge its silt in the reservoir. Because of greater specific gravity and lower temperature, it may pass through the reservoir as an underflow and be discharged over the dam with its burden of silt. Such underflows are called "density flows." They have been experienced in many large reservoirs. If their control were understood the silting problem might be less of a menace. One method of control might be to release silt-bearing waters before their silt is deposited.

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[1] BROWN, C. A., "Reservoir Silting," *Water Works & Sewerage*, June, 1941, p 257.

[2] WITZIG, B. J., "Sedimentation in Reservoirs," *Trans. A.S.C.E.*, Vol. 109, p. 1047, 1944, with bibliography.

After silt has been deposited in a reservoir, there are no practicable methods, widely applicable, for removing it. Dredging is expensive, and the disposal of the dredged material presents a serious problem. The opening of sluices in the dam near the bottom of the reservoir is ineffective, as a negligible portion of the silt will be moved. Silting must be avoided; it is cured only infrequently. It may be avoided by site selection, erosion prevention, and desilting works. The reservoir site may be chosen on a non-silt-bearing stream, or the reservoir may be located in a basin off the main channel so that heavily silt-laden waters may be by-passed around the basin. Reservoirs should be located on the smallest possible drainage areas, because it has been shown that the rate of silting increases directly as the ratio of the drainage area to the reservoir capacity. Witzig recommends a reservoir capacity of at least 75 acre-ft. storage per square mile of drainage area for a 100-year life of typical reservoirs in eastern United States. Erosion prevention methods recommended for soil conservation include proper crop rotation, plowing on contours, terracing, strip cropping, protected drainage channels, check dams, reforestation and fire control, and grazing control. Desilting works consisting of mechanically cleaned sedimentation basins may remove silt from the water before it enters the reservoir. The silt removed is discharged into the stream below the dam.

202. Reservoir Problems. Problems resulting from the construction of an impounding reservoir are many and, in some cases, are related. Among a few of these problems may be included (1) the effect on public health in the region,[1] (2) the area of land to be flooded, (3) the type and capacity of the spillway, and (4) the routing of floods through the reservoir. Problems affecting the public health may include (1) malaria control resulting from increased mosquito breeding, (2) recreational problems resulting from increased water surface; (3) fishing and hunting, (4) food supplies and agriculture, (5) industrial developments, (6) safety, and (7) drainage. Construction and control of the reservoir to protect the water quality are discussed in Secs. 448 and 450. In general, it is desirable to avoid shallow water and to avoid alternately exposing and inundating large areas along the shore. Steep-sided reservoirs are, therefore, ideal but are not always obtainable.

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[1] See also JONES, S L., and A. WOLMAN, "Public Health Problems in Impounding Water in the Tennessee Valley," *Jour. A.W.W.A.*, April, 1945, p. 327.

Spillways are discussed in Chaps. II and VII. In general, they should be designed to minimize the fluctuation of water level between spillway-crest level and high-water level. The determination of the surface elevation in the reservoir during flood or when water is being withdrawn from the reservoir involves tedious computations.

203. Routing of Floods.[1] By the routing of a flood through a reservoir is meant the determination of the rise in elevation of the water surface in respect to time. Posey suggests the following method: Select a small increment of time t of such duration that the inflow may be assumed to be constant. Satisfactory accuracy may be obtained if there are 15 or 20 steps during the time of a single flood peak.

Let S_1 = storage at the beginning of time
S_2 = storage at the end of time t
i = average rate of inflow
o = rate of outflow

Then

$$S_1 + it = S_2 + ot \qquad \text{and} \qquad it + \left(S_1 - \frac{o_1t}{2}\right) = \left(S + \frac{o_2t}{2}\right)$$

where o_1 is the outflow corresponding to S_1, and o_2 is the outflow corresponding to S_2. A timesaving computer for the solution of reservoir problems is described by Shepley and Walton.[2]

From these formulas, together with a depth-capacity curve and the calibration of the spillway, a time-surface elevation curve can be plotted for the routing of any flood peak through the reservoir.

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[1] See also POSEY, C. J., "Routing of Floods through Reservoirs with Especial Attention to Determination of Spillway Capacity for Small Reservoirs," *Bull. Assoc. State Eng. Soc.*, October, 1938, p. 155.

[2] *Civil Eng.*, March, 1942, p. 154.

CHAPTER XII

PUMPING STATIONS

204. Buildings.[1] The architecture and surroundings of a waterworks pumping station can be made as attractive as desired, and the station can be operated without nuisance to the neighborhood. A building of attractive exterior and interior design, with the grounds attractively landscaped and well maintained, to which the public is made welcome, will be justified by public confidence, esteem, and good will. Fireproof materials should be used throughout the structure. Two attractive fireproof pumping stations are illustrated in Figs. 95 and 96, the former being suited to a residential district and the latter to an industrial district. It is obvious that the building must be large enough for and suited to the purposes of a waterworks pumping station. Possibilities for growth and expansion must be considered in the original design.

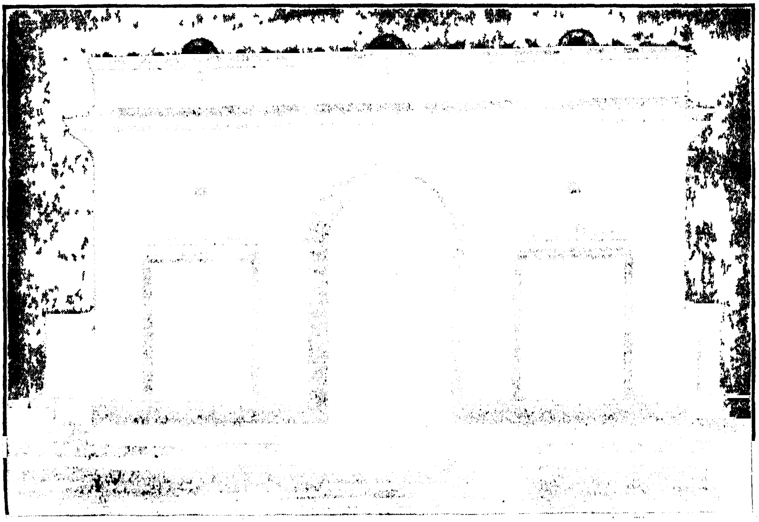

FIG. 95—Thomas Jefferson pumping station, Chicago, Ill. (*Courtesy of L. D. Gayton*)

The design of the interior of the building includes the consideration of such factors as space requirements for each piece of equipment; the

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[1] See also "Tentative Manual of Safe Practice in Water Distribution," *Jour. A.W.W.A.*, June, 1942, pp. 917 and 926.

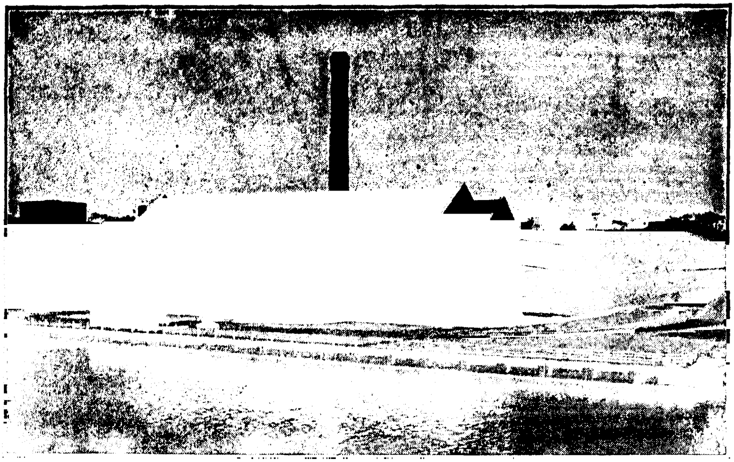

FIG. 96.—Riverside pumping station, Milwaukee, Wis. (*Courtesy of F. Krieger.*)

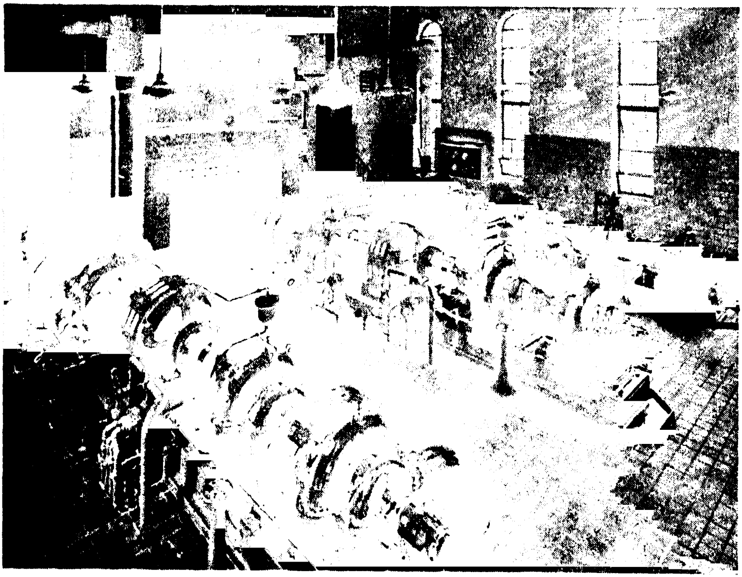

FIG. 97.—Steam-driven centrifugal pumps, Newton, Mass. (*Courtesy of De Laval Steam Turbine Co.*)

proper location of the equipment; the heating, lighting, ventilation, and drainage of the building; and the interior decoration. Space requirements of equipment must be determined from its design or, in the case of "stock" equipment, from catalogue descriptions or similar information. The net space requirements will be determined in this manner. The total space allowed should provide for accessibility and convenience. A convenient interior arrangement of a small waterworks pumping station is shown in Fig. 97 and of larger pumping stations in Figs. 98 and 99. Equipment in a small, dark, unpleasant location is more easily neglected than equipment accessible for observation, operation, and repairs. The location of a piece of equipment is more directly affected by the requirements of accessibility and convenience than it is by space requirements. Equipment should be located so as to provide short well-laid-out steam piping, electric wiring, belt connections, or other methods of power transmission. Each piece of equipment should be accessible to an overhead crane or so located that it can be removed or replaced without disturbing other pieces of equipment. The interior arrangements of two pumping stations are illustrated in Figs. 98 and 99.

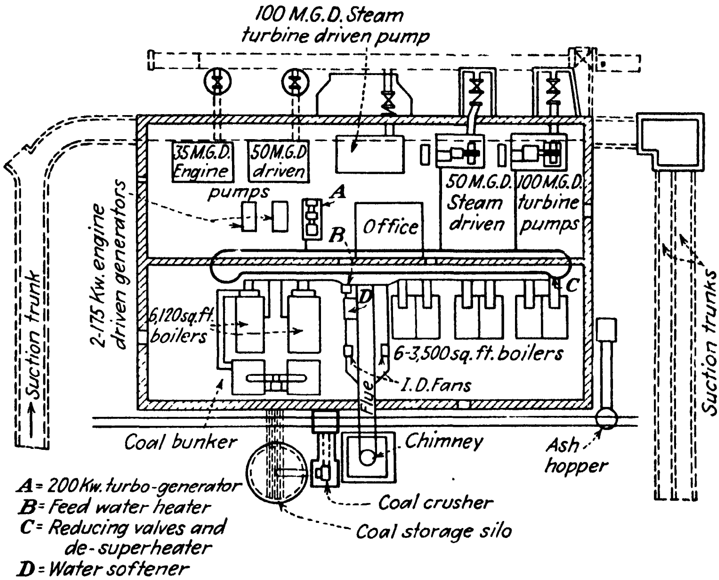

FIG. 98.—Interior arrangement, Ross pumping station, Pittsburgh, Pa. (*From Power*, *Apr.* 23, 1929.)

Provision should be made for adequate heating for the comfort of the operators and for the safety of the equipment. The ventilation and lighting of all parts of the building should be generous so as to assure that no equipment will be neglected, to serve as an incentive to careful operation, and to create pride on the part of the operators in the appearance and maintenance of the equipment. A dark, dusty, gas-filled boiler room or a damp, poorly lighted engine room is unnecessary and avoidable. Plumbing in washrooms must be located and installed to avoid possibilities of contamination of the water. All surface drainage must be led away from the water intake or wet well.

The interior decoration should be simple, light colored, and dignified, in keeping with the important nature of the service being rendered. Interior walls of white or light-colored glazed tile with dull-colored floors of tile or other ceramic or nonabsorbent material are frequently used and are satisfactory.

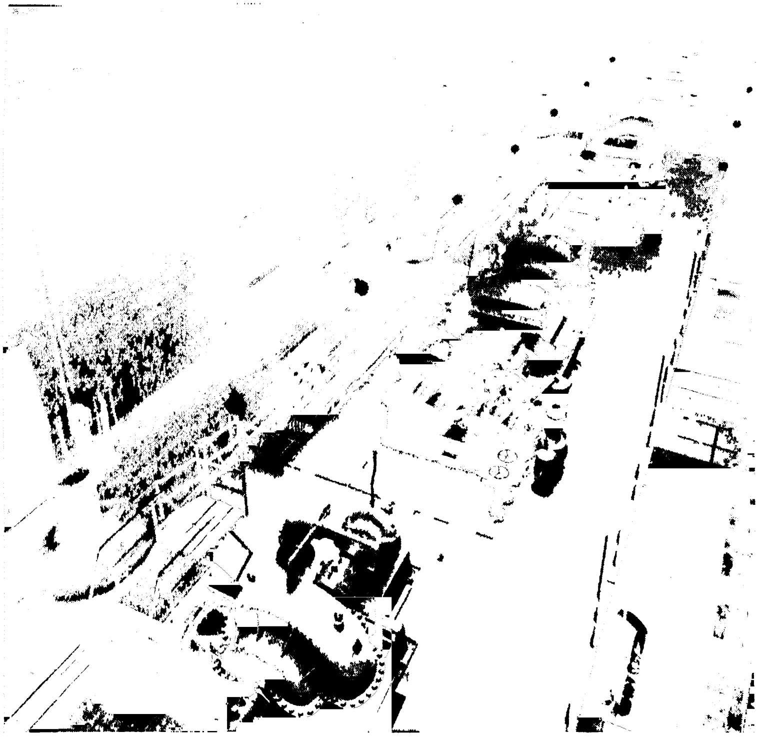

FIG. 99.—Pump room, Western Ave. pumping station, Chicago. (*Courtesy of De Laval Steam Turbine Company*)

205. Location. Factors to be considered in the location of a waterworks pumping station include (1) sanitary protection of the quality of the water; (2) the hydraulics of the distribution system; (3) possibilities of interruption by fire, flood, or other disaster; (4) the availability of power or fuel; (5) growth and expansion; (6) and the attitude of the neighboring property owners. Floods offer a hazard which may be minimized by favorable location. To obtain the greatest hydraulic advantage, the station should be located near the middle of the distribution system. The danger of interruption of service by fire in the immediate neighborhood of the pumping station should be considered and a site chosen to avoid or minimize this hazard. Where it is impossible to find such a protected site, the station should be protected by fire walls, fireproofing, and a sprinkler system.

The use of electricity permits the location of the pumping station at any desired site. If coal is to be used, the station should be located near a railroad or waterway so that coal may be readily transported to it. In general, pumping stations located in residential districts should be operated by electricity, as it is difficult and expensive to suppress all the nuisances in connection with the operation of a steam plant.

There should be no reasonable objection to the location of a waterworks pumping station in any district, provided pleasing architecture and the proper operation of the station are assured. A waterworks pumping station in a residential district is shown in Fig. 95, and one in an industrial district in Fig. 96. It is obvious that the style of architecture should conform to the character of the district in which the pumping station is located.

206. Choice of Power. Steam, electricity, water, wind, and fuels consumed within the engine are all used for driving pumping engines. Steam, electricity, and internally consumed fuels are commonly used in pumping public water supplies. The type of power selected should be the most reliable, the most available, and the least expensive. If all three conditions cannot be fulfilled, they should be rated in the order stated, with the greatest reliability as the most important. "All main pumping stations should be provided with auxiliary sources of power sufficient to maintain operations in case of interruption of the primary power source . . . ."[1]

The design of the largest pumping station usually leads to the selection of steam power generated and applied at the pumping station. In cities under 50,000 to 100,000 population, and for auxiliary pumping stations in larger cities, electrically driven pumps are commonly installed. Because of the possiblity of the failure of the supply of electric power, particularly when it is imported from an outside source, it is desirable to install standby power equipment, either steam boilers and engines or internal-combustion engines. The engines may drive the pumps directly or may be used to drive auxiliary electric generators. Gas or gasoline engines, because of their quick-starting characteristics, are suitable for auxiliary power. The use of internal-combustion engines as a primary source of power is apparently growing in favor, particularly in the case of the Diesel engine.

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[1] From "Tentative Manual of Safe Practice in Water Distribution," *Jour. A.W.W.A.*, June, 1942, pp. 917 and 927.

The centralization of the generation of electric power is resulting in the availability of current at a low price—less than 1 cent per kilowatt-hour in some localities. Another saving in electrically driven equipment is the reduction in first cost compared with steam power, the fixed charges thus being reduced compared with those required for the equipment and buildings needed for steam-power generation and application.

207. Classification of Pumps. Pumps may be classified as steam, electrical, water power, wind power, etc., in accordance with the kind of power used to actuate them. They may be classified as well pumps, low-service pumps, high-service pumps, etc., in accordance with the kind of service to which they are to be put. Low-service pumps operate with relatively low discharge pressures, and high-service pumps with high discharge pressures. Pumps may be classified also as displacement, centrifugal, impulse, or bucket pumps, in accordance with the mechanical principles of their operation.

208. Capacity of Pumps. A waterworks pumping station should be designed to supply the peak demand without dangerously overloading the power and pumping equipment. It is not necessary that the pumping-station equipment should carry the peak demand at rated capacity. Practically all types of power and equipment will operate successfully for various periods of time under some overload. The percentage of overload, the period of time this overload can be carried, and the rated capacity of the unit depend on the type of the equipment. For example, a water-tube steam boiler may stand 250 per cent overload indefinitely with proper coal-burning equipment, but most types of electric motors would fail under more than half such a percentage of continuous overload.

The highest efficiency and the greatest economy of operation demand that the total load on all the units in the pumping station should be divided among them in such a manner that each can operate at its rated capacity. The operation of a pump or engine at other than rated capacity may be uneconomical because of the lower efficiency of the machine at other than full-load conditions.[1] Units are available that will operate efficiently when either underloaded or overloaded.

If electrical equipment is used in the pumping station and electric current is manufactured by the city or the waterworks department, then sufficient capacity must be provided in the power plant to operate the pumps in an emergency at their maximum capacity. The maintenance and fixed charges on this additional equipment are items in the expense of the waterworks. If electrical power is purchased from an independent manufacturer, such fixed charges are avoided, but the power company may be justified in levying a charge for readiness to serve if excess current is quickly available, whether or not such current is used. The ready-to-serve charge may be minimized or avoided by arranging a proper operation schedule for the pumping station.

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[1] See also TULL, E. R., "Operating Economies Effected through Installation of Smaller Pumping Equipment," *Jour. A.W.W.A.*, April, 1935, p. 454.

"Pumping station units and auxiliary equipment should be of ample capacity and in sufficient numbers of units to permit at least one unit being out for repairs without seriously reducing the station capacity."[1]

209. Selection of Pumps. In the selection of waterworks pumping equipment considerations of capacity, lift, reliability, and economy are paramount and should be rated in the order indicated. Additional factors include conditions of suction lift, variations in load, floor-space requirements, flexibility of operation, starting and priming characteristics, type of drive most suitable, and other factors.[2]

The centrifugal pump, because of its outstanding advantages, has almost entirely displaced other types of pumps in recent waterworks practice. The crank-and-flywheel steam pumping engine, which reigned supreme until recently, has now definitely been displaced. The centrifugal pump most satisfactorily fulfills the requirements of waterworks service.

Considerations of operating efficiency are important economically. For example, assume that $1,300 will buy a motor and a 2,000-g.p.m. centrifugal pump with a total lift of 150 ft. operating with 70 per cent load factor and an over-all efficiency of 75 per cent. If the cost of electric power is 0.45 cent per hp.-hr., life is 15 years, and interest is 3.5 per cent, an increase of 2 per cent in the over-all efficiency will be worth $1,420, slightly more than the first cost of the equipment.

210. Piping. The layout of the piping in a pumping station may be as important as the efficiency of the pumping equipment in affecting the economy of station operation. Short, straight, well-supported pipe lines, devoid of traps either for sediment or for gas, sloping in one direction to drains, and with adequate cleanouts and valves should be the object of the designer. Desirable velocities may be estimated from a study of Table 43 with the knowledge that the economical velocity in pipes used infrequently is higher than the figures shown in the table. A few details to be observed in making pipe connections to centrifugal pumps are as follows: (1) The elbow on the suction side of a double-suction pump should be vertical to the suction nozzle, or a special elbow with guide vane followed by a piece of straight pipe to the suction nozzle should be used, to equalize the flow on each side of the impeller. (2) On the pump discharge, where change of direction is necessary, use a constant-diameter bend, followed by a long straight reducer or increaser. Such an installation will most effectively interchange velocity and pressure heads. Eccentric reducers with the top of the pipe and the top of the reducer at the same level should be used in horizontal suction piping to avoid the creation of an air pocket. An increaser whose length is ten times the difference in diameters may be considered as "long."

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[1] From "Tentative Manual of Safe Practice in Water Distribution," *Jour. A W. W.A.*, June, 1942, pp. 917 and 922.

[2] See also BINDER, H. P., "Pumps with Special Attention to Water Works Practice," *Bull. Assoc. State Eng. Soc.*, July, 1934, p. 33; and MAXWELL, W. H., "Water Supply Problems and Developments," Sir Isaac Pitman & Sons, Ltd., London, 1934.

211. Surge Control. Surge or water hammer in pumping stations and pipe lines result from starting or stopping of a pump or pumps, sudden opening or closing of a valve of any type on a pipe line in which water is moving, the formation of a partial vacuum in a pipe and its sudden release, and other causes. The effect of the phenomenon of surge or water hammer is to produce pressures either above or below normal that may rupture the pipe or draw in pollution. Methods of control[1] include the use of (1) a flywheel on the pump which, by adding inertia, slows its starting or stopping; (2) motor-driven gate or stop valves that control the rate of closing of the valve; (3) electrical resistances in the starting mechanisms of the electric motor, slowing the rate at which the motor may be accelerated or decelerated, (4) a pressure-relief valve; (5) a surge suppressor, similar to a pressure-relief valve, except that the valve is opened by a partial vacuum instead of by a plenum; (6) an air chamber; (7) a surge tank; and (8) a surge buffer which consists of a closed vessel with an air valve on top. The vessel is normally filled with water. When a pressure below atmospheric is experienced, the air valve opens and admits air, which acts as a cushion on the arrival of a pressure wave. When normal conditions are restored, the air is allowed to escape from the vessel which refills with water.

212. Suction Lift.[2] Suction lift should be avoided, where possible, by installing the pump either in a wet well or in a dry well below the level of the water in the intake. The location of pumps in inaccessible pits or dry wells subject to inundation is objectionable for reasons of sanitation and maintenance. Motors driving the pumps should be located above the water level in the intake.

Where suction lift cannot be avoided, provisions should be made for priming the pumps, or vacuum pumps may be installed to exhaust air from the suction lines. Care should be taken to assure the use of only potable water for priming.

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[1] See also LAPWORTH, C. F., "Control of Surge in Pipe Lines," *Jour. A.W.W.A.*, October, 1945, p. 1036.

[2] See also "Tentative Standards of Safe Practice in Water Distribution," *Jour. A.W.W.A.*, June, 1942, pp. 917 and 925

Suction lines should be made as short and as straight as possible with a continuous upward slope from the foot valve in order to avoid air pockets. To minimize suction lift it is customary to design the suction pipe larger than the discharge pipe on the pump.

The height to which water can be raised by suction is limited by the pressure of the atmosphere, the temperature of the water, the velocity in the suction pipe, and the friction losses. It can be expressed as

$$H = A - (V_p + V_h + H_f)$$

where H = possible suction lift
A = pressure of atmosphere
V_p = vapor pressure of water
V_h = maximum velocity head in suction line
H_f = sum of the friction losses in suction pipe

213. Operating Schedules. Waterworks pumping stations are usually operated continuously. In the smaller communities with a population under 5,000 to 10,000, it is sometimes economical to provide pumping equipment and storage reservoir capacity so that the pumps need be operated only a part of each day, thus reducing the operating force to one shift. In cities up to 30,000 to 50,000 inhabitants, fire pressure may be supplied by the pumps in the pumping station, thus avoiding the need for fire engines. This affects not only the design of the pumping equipment but also the organization and alertness of the operating force.

214. Auxiliary Pumping Stations. Auxiliary pumping stations are sometimes installed on large distribution systems to increase the pressure which otherwise might be too low. Low pressures in a district may be caused by inadequate sizes of pipe; a large localized consumption, as in an industrial section; or high elevations above the main pumping station. Auxiliary pumping stations are sometimes equipped with automatically operated devices which may be remotely controlled at a central station. Steam power is not suited to automatic control; an automatically controlled, internal-combustion engine should not be left operating without attention. Electric motors and equipment are most suitable to unattended automatic operation or remote control.

Automatic or remote control is convenient, and it may be economical. No full-time operator is needed at the auxiliary station where electric equipment is installed. Daily, or more frequent, visits are desirable, however, and dependence upon automatic apparatus may result in costly breakdowns due to neglect, improper lubrication, or other causes.

215. Cost of Water and of Pumping. The cost of water is markedly different in different communities because of the various conditions under which it must be obtained. The least expensive supply is a potable surface supply which can be distributed without pumping and is available near the city. Ground waters, at a remote distance, requiring purification are among the most costly. It may cost anywhere from 1 cent to 30 to 40 cents, or more, per 1,000 gal. of water delivered to the customer to construct and maintain a public waterworks.

The cost of pumping water includes interest and depreciation on the building and equipment, sometimes called the fixed charges; and the wages of labor, the cost of power, insurance, taxes, etc., sometimes called the operating charges. In addition to these items the cost will be affected by the efficiency of the pumping equipment and the rate of interest allowed in amortizing the plant. For comparative purposes the cost should be expressed in cents per million gallons raised 1 ft. In practice it is customary to express the cost of water in cents per thousand gallons, regardless of the lift.

CHAPTER XIII

STEAM AND RECIPROCATING PUMPING MACHINERY

216. Pumping-station Equipment. Improvements and economies in the generation and application of steam power have so kept pace with progress in internal-combustion engines and the centralized production and transmission of electric power that, under favorable conditions, the development of steam power and the use of steam-driven pumping machinery, particularly in the larger installations, will prove economical. The reciprocating pumping engine is considered obsolete. Some knowledge of its characteristics is desirable, however, as the machine is in use, and its possibilities should be appreciated.

Recent developments in steam pumping-station design include a marked increase in steam pressure; the use of economizers; air preheaters; better furnaces and combustion equipment; the use of electric drive for station auxiliaries; the use of the regenerative cycle for feed-water heaters; and improvements in turbines, condensers, and auxiliaries. Some of the latest steam plants have eliminated the economizer, finding it more economical to salvage the heat for air preheating and to heat the feed water with live steam.

Furnace and boiler efficiencies of 85 per cent are commonly attained[1] and under test conditions are over 90 per cent. This has been accomplished at the same time that lower stacks have been made possible by the more extensive use of forced draft and mechanical stokers.

Recording instruments, such as rate of pumping, water discharge pressures, total lift, coal weighing, feed-water measurements, steam pressure, flue-gas temperature, feed-water temperature, electric consumption, etc., should be included in the equipment of a pumping station. The existence of such continuous records, provided by such equipment, supplies valuable data for operation and serves also as an incentive to efficiency on the part of the operators.

217. Steam Pumping Engines. A steam pumping engine, or, as more commonly known, a "steam pump," consists of one or more steam cylinders with pistons which are connected directly or by cranks and shafts to one or more water cylinders. The water end of a horizontal, outside center-packed plunger pump with condenser is illustrated in Fig. 100.

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[1] See also EYRE, T. T., "Recent Advances in Steam Plant Design," *Civil Eng.*, September, 1935, p. 583.

Steam pumping engines are classified in accordance with the number of steam and water cylinders, the expansion of the steam, and other conditions. A pump taking steam from the boiler during the full length of the stroke is known as a direct-acting steam pump. Such pumps are used in small waterworks pumping stations with capacities up to 2,000,000 gal. per day.

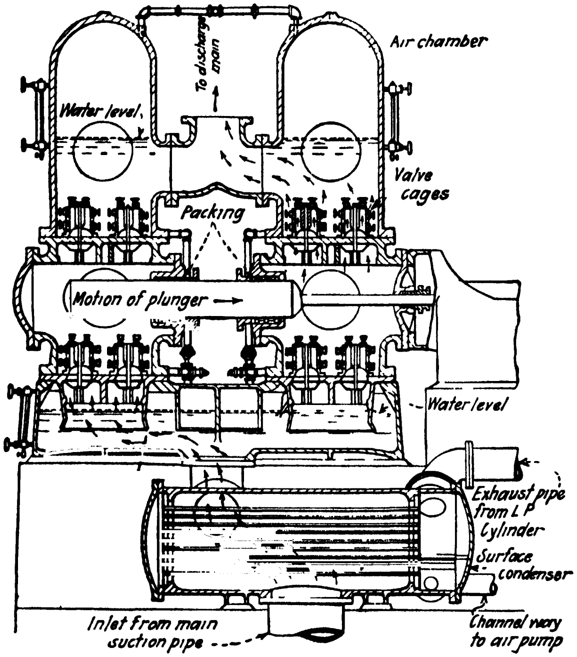

FIG 100. Water end of pumping engine. (*Courtesy of Allis-Chalmers Manufacturing Co.*)

A triple-expansion steam engine is illustrated in Fig. 101 with a steam turbine-driven centrifugal pump in the foreground. Although the steam engine may be equally efficient or more so than the turbine-driven centrifugal pump, its first cost, maintenance cost, and the capital charges on the larger building required to house the pumping engine have combined to restrict the use of this type of pumping machine.

218. Steam Turbines. The steam turbine is a prime mover using the expansion and the velocity of steam to generate power. Its efficiency and simplicity are secured, in part, through the absence of reciprocating parts and ease in lubrication.[1]

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[1] See also CHURCH, E. F., "Steam Turbines," McGraw-Hill Book Company, Inc., New York, 1935.

A steam turbine is simpler in construction and is easier to operate than a high-duty triple-expansion pumping engine. Turbines are adaptable to the use of steam at pressures varying from less than atmospheric to the highest pressures and degrees of superheat practicable in steam boilers. Over-all heat efficiencies of 18 per cent are not uncommon, and 22 per cent has been attained.[1] Water rates of 12 to 20 lb. of steam per indicated horsepower-hour can be anticipated with their use. The smaller rates are obtained in the larger units. Owing to low first cost, relatively small space requirements, and efficient operation, the total charges on a steam-turbine centrifugal-pump unit are usually lower than on a triple-expansion pumping engine. The result is that in the latest installations in large pumping stations there is a tendency to install the turbine-driven pumping unit.

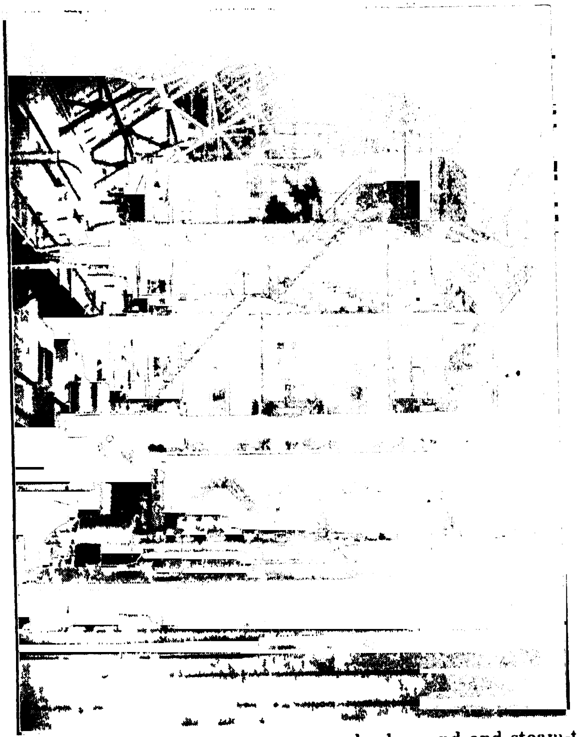

FIG 101.—Triple-expansion pumping engine in background and steam-turbine-driven centrifugal pump in foreground. Riverside pumping station, Milwaukee, Wis. (*Courtesy of Ferdinand Krieger*.)

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[1] See "Mt. Royal Pumping Station, Baltimore," *Jour. A.W.W.A.*, Vol. 10, p. 758, 1923.

Steam turbines are suited particularly to the driving of centrifugal pumps or electric generators. An installation is illustrated in Fig. 97. Because the economical speed of steam turbines is much higher than the economical speed of centrifugal pumps, the two must be connected in such a manner that the pump may revolve more slowly than the turbine. This is commonly accomplished by means of reduction gearing. Steam turbines require relatively small housing space compared with reciprocating steam pumping engines, and they can be constructed in any size. Units of 100,000 hp. have already been constructed, and there seems to be no limit to their capacity. An added advantage to their use is the extreme flatness of the rating curve. A test at Cleveland[1] showed a variation in power consumption of 7 per cent for a load variation of 100 per cent.

Practice has found it best to use superheated steam in turbine operation, but as the pressure drops in the turbine the steam must be reheated. For this purpose exhaust steam from the turbine is led through reheaters and is then used to drive the turbine. An advantage of bleeding steam from the turbine to preheat feed water is that it reduces the volume of low-pressure steam that must be handled in the turbine and, at the same time, utilizes the heat efficiently.

219. Water Rates and Duties. The water rate of a steam engine is the number of pounds of steam consumed per indicated horsepower-hour. The most efficient machines may be expected to have water rates of 12 to 18 lb. Some less efficient machines, as direct-acting noncondensing steam pumps, may have rates as high as 50 to 80 lb. An indicated horsepower-hour is a measure of the energy delivered by the steam to the engine. It must not be confused with the horsepower delivered by the engine. The mechanical efficiency of an engine is the ratio of the delivered, or brake, horsepower to the indicated horsepower.

The word "duty," used as a measure of efficiency, has been applied mainly to pumping engines. Duty and efficiency are expressions of the same thing in different units. The *duty of a pump* is the number of foot-pounds of work done by the pump per 100 lb. of coal, or per 1,000 lb. of steam, or per 1,000,000 B.t.u. consumed in doing the work. These units are only approximately equal, with the result that there may be some discrepancy in the measure of the duty of a machine unless it is measured in terms of heat units. A high-duty pump should have a duty above 150,000,000. An ordinary pumping engine may be expected to have a duty of 100,000,000. The duty of a pump, expressed in British thermal units divided by 778,000,000, is equal to its over-all efficiency.

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[1] See also MULLERGREN, A. L., "Modern Pumping Station Design and Its Probable Future Development," *Jour. A.W.W.A.*, Vol 18, pp. 180 and 184, 1927.

220. High-pressure and Superheated Steam. High-pressure and superheated steam are used to increase the efficiency of power production. The limit of the pressure and the temperature is fixed by the strength of available materials and the lubrication problems developed. At a waterworks pumping station it is not customary to exceed 500 p.s.i. pressure with 250° of superheat.

L. A. Day stated that:[1]

> Superheated steam in the cylinder of a steam engine transfers heat to the cylinder walls but instead of condensation occurring on contact as with saturated steam, the superheat must first be withdrawn. Superheated steam maintains a higher cylinder-wall temperature and, if superheating is carried far enough, condensation can be delayed until after cutoff and even until expansion is partly completed. Also, the specific volume of superheated steam is greater than that of saturated steam, and this increase in specific volume, the pressure being constant, diminishes the weight of steam to the engine and has an influence on the economy gain, as has also the lower thermal conductivity of superheated in comparison with saturated steam.

Owing to the absence of moisture, superheated steam offers less friction passing through pipes than is offered by saturated steam. Therefore the speed of travel can be higher, and the piping, fittings, and valves can be smaller. When piping is designed specially for superheated steam, the reduction in size will go far toward paying for the superheaters. Where superheated steam is turned into existing piping, the radiation may be somewhat greater than if the piping were the proper size, but the reduction in pressure would be less. The actual radiation from superheated steam, under similar conditions, is less than it is with saturated steam, because, even though the temperature is higher, its low conductivity makes it lose heat more slowly. Besides, the pipes are not wet and so have less conductivity. Depending on the steam speed and the pipe protection, superheated steam is figured to lose 1° of superheat in 6 to 10 ft. of travel.

221. Overload and Underload. The power that a pump will deliver when operating under the most economical conditions should be the rated power of the pump. This is usually the load for which the pump will operate at maximum efficiency, *i.e.*, with lowest water rate. Operation at the lowest water rate may not give maximum over-all economy, as the operation at a faster rate may reduce the cost per unit of volume pumped by spreading the first cost over a larger amount of water, or operation at a slower rate may increase the life of the pump. The water rates of various types of pumping engines at various degrees of over- and underload are shown in Fig. 102.

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[1] DAY, L. A., "Some Economic Features of Pumping Station Operation," *Jour. A.W.W.A.*, Vol. 7, p. 795, 1920.

Operation at any but the rated load is usually less economical than operation at rated load. Part-load operation should be avoided where possible in routine operation. In the selection of pumps for a particular load, if dependence is placed upon carrying peak loads on the pumping station at maximum permissible overload on the pumps, the first cost of the pumps will be lowered. The maximum overload to which a pump can be subjected is dependent on the design and construction of the pump and on the duration of the overload. It is unsafe practice to depend on more than about 150 per cent overload capacity for a long time.

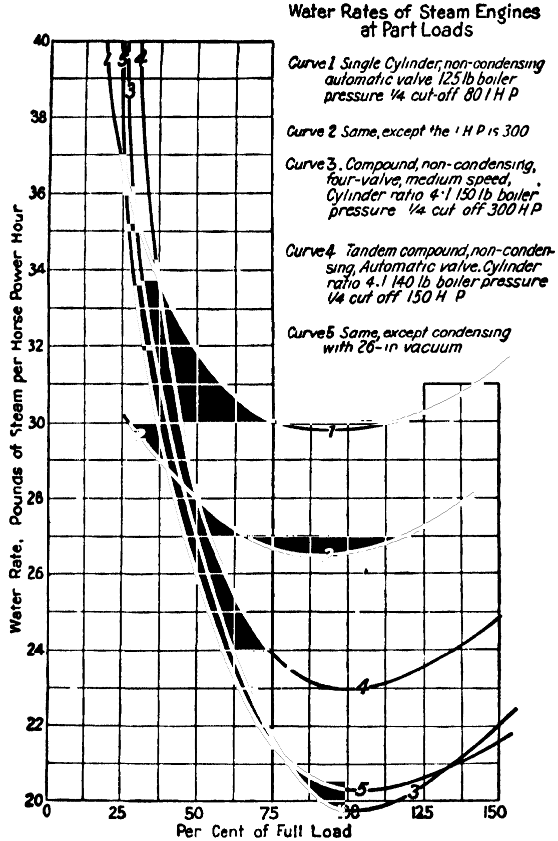

FIG. 102.—Water rates of steam engines at part loads.

222. Compensators. The force that causes the discharge of water from a reciprocating steam pump should be applied at a constant rate during the entire stroke in order to balance the resisting forces which are normally constant. If the total pressure on the steam end of the pump drops below that on the water end, the movement of the piston will be stopped or reversed by the water pressure.

When steam is used under full boiler pressure for the full length of the stroke, no difficulty is encountered if

$$A_sP_s = A_wP_w$$

in which A_s and A_w are the areas of the steam and water pistons, respectively, and P_s and P_w are the intensities of the steam and water pressures, respectively. If the steam pressure is cut off from the boiler at some time during the stroke and steam is allowed to expand, the pressure drops immediately and motion would cease if other forces were not exerted. Any device for exerting such a force to carry the pump through the remainder of the stroke after cutoff is called a "compensator." The compensator absorbs energy when the steam port is open, and it delivers energy to the pump when the steam is cut off. Various ingenious devices have been invented and used for compensation, but the one device in almost universal use at the present time is the flywheel.

223. Waterworks Type of Condenser. The maximum possible expansion of steam is secured by discharging into a vacuum. The desired vacuum is usually created by the condensation of the exhaust steam from the steam cylinder. The use of a condenser cuts down the back pressure on the exhaust and makes additional pressure available for driving the pump. The amount of additional pressure is dependent upon the intensity of vacuum maintained. The intensity of vacuum is usually expressed in inches of mercury.

Two types of condensers frequently used in power plants are the surface condenser and the jet condenser. Surface condensers are either water- or air-cooled. In a waterworks pumping station, water-cooled surface condensers are used almost exclusively. In the water-cooled surface condenser, steam is condensed by passing it through tubes which are surrounded by the cooling water, as shown in Fig. 103. This type of condenser is so commonly used in waterworks pumping stations that it is frequently called the "waterworks type." Such condensers are so designed that the cooling tubes have about 2 sq. ft. of cooling surface per horsepower of the pump.

A vacuum as high as 25 in. of mercury, which is equal to an absolute pressure of about 2 p.s.i., is ordinarily carried in a waterworks condenser.

A vacuum pump for the removal of air from the condenser is essential to its successful operation.

The waterworks type of condenser is commonly connected in the suction line of the pumps so that the entire public water supply is used for cooling the condenser. The rise of temperature of the water is so slight as to be unobjectionable.

224. Steam-pump Auxiliaries. The auxiliary pumps and motors in a steam pumping station include the feed-water pumps, circulating pumps, and other condenser auxiliaries; oil filters; mechanical stoker drives; and draft mechanism.[1]

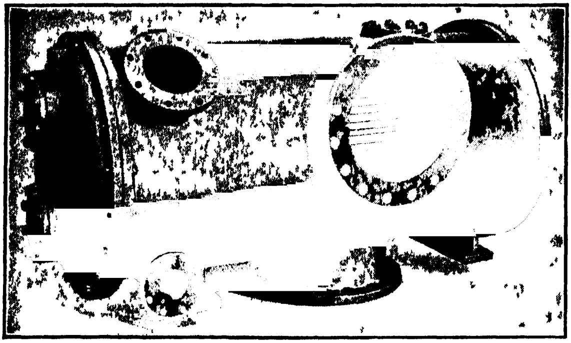

FIG. 103. Waterworks type of surface condenser. (*Courtesy of Allis-Chalmers Manufacturing Co.*)

Recent practice is to use electric drive for all auxiliaries, if possible installing a d-c electric generator on the shaft of the turbine-driven main pumping unit. For starting up the plant, some stand-by unit will be required. As such an arrangement prevents the use of exhaust steam for feed-water heating, it has been found more economical to bleed live steam directly into the feed water. This is called the regenerative system.

225. Pistons and Plungers. Water may be drawn into or expelled from the water cylinders of a pumping engine by either pistons or plungers. A piston is a diaphragm which fits close to the walls of the cylinder and expels or draws in water by displacement. Packing is placed on the outer edge of the piston so as to expand and touch the walls of the cylinder, thus preventing the passage of water from one side of the piston to the other.

A plunger is illustrated in Fig. 104. It is to be noted that it does not fit tightly against the walls of the cylinder. As the plunger moves in and out of the water cylinder, water is drawn in and expelled. The packing used to prevent the passage of water around the plunger is shown at *A*. This packing is in an advantageous position, as it is visible and accessible to the operator who can adjust it as desired without stopping the operation of the pump. A plunger pump is more suitable than a piston pump for the handling of water carrying gritty material.

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[1] CUNNINGHAM, F. G., "Selection of Auxiliaries for Steam-operated Pumping Stations," *Jour. A W W.A*, March, 1924; FERNALD and ORROK, "Engineering of Power Plants"; "Serial Report on Condensing Equipment," Prime Movers Committee, *Nat. Electric Light Assoc., Pub* 24-78.

226. Speed and Stroke of Pumps. The stroke of a pump is the linear travel of the water piston or plunger from one extreme position to the other. The reversal of motion consumes energy; hence, long-stroke pumps lose less energy than short-stroke pumps per unit of volume pumped, but the greater length of steam and water cylinders adds to the first cost of the pump.

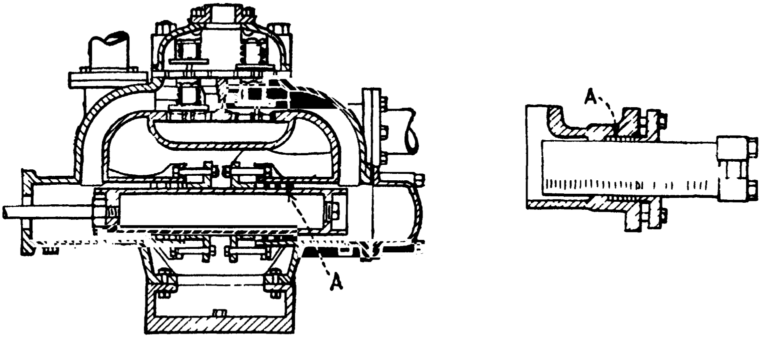

Fig. 104 Center-packed plunger pump and an end-packed plunger *(From Standards of the Hydraulic Society)*

The speed of travel of the piston or plunger is limited to about 100 to 150 f.p.m. where "ordinary" water valves are used. This is because the valves do not close with sufficient rapidity to permit greater speeds without too much slippage of water past the valves. The slippage of a pump is the amount of water that passes from one side to the other of the piston or plunger plus that which passes the valves in the wrong direction. Slippage is usually expressed as a percentage of the total displacement of the piston or plunger. For example, if it is found that a pump with a 10-in. piston, a 2-in. piston rod, and a 12-in. stroke, running at 50 r.p.m., delivers 375 g.p.m., the slippage will be $(400 - 375)/400$, which equals 6 per cent slippage. The denominator of this fraction represents the total displacement of the pump in gallons per minute.

Slippage through the water valves due to high speeds of operation can be partly overcome by moving the valves with mechanism directly connected to the moving parts of the pumping engine. With such equipment, piston or plunger speeds up to 500 f.p.m. are used.

227. Water Valves. An ordinary water valve for a pump is shown in Fig. 105. The valve consists of a leather or rubber disk that rests across the ports through which water passes. The disk is held in position by a coiled spring held by a spindle threaded into the center of a "spider," shown in the figure. The valve is opened by the pressure of the water beneath it and is closed by the water and the spring when the water attempts to pass in the reverse direction. The diameter of the disk is limited by the pressure against which the pump operates, but valves more than about 4 in. in diameter are not used because of the danger of breakage of the disk. The desired area of port openings is secured by using a large number of valves. They are grouped on "cages," as shown in Fig. 106. The total area of the port opening should not be less than about twice the area of the suction or the discharge pipe served.

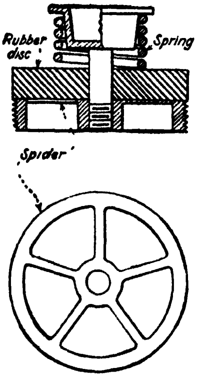

Fig 105 --Ordinary pump valve

228. Foot Valves. Foot valves are used on the lower end of the suction pipe of a pump to prevent water that has already entered the suction pipe from falling back into the suction well. One type of foot valve is illustrated in Fig. 107. The net area of the opening of the valve should exceed 150 per cent of the cross-sectional area of the suction pipe.

Foot valves are necessary on single-acting pumps and on ordinary centrifugal pumps that are not operating submerged. They are used on other types of pumps as a measure of protection. Where foot valves are used on large pumps with high suction lift, the water hammer caused by sudden closing of the foot valve may injure the pump. This difficulty may be overcome by placing a check valve in the suction pipe close to the pump. The check valve should be designed to close slightly in advance of the foot valve.

Fig. 106 — Pump-valve cage complete. Standard horizontal and vertical (*Courtesy of Allis-Chalmers Manufacturing Co*)

The foot valve is usually protected by a screen to prevent the entrance of material that might clog or injure the pump. The cleaning of the screen may require frequent attention under certain operating conditions.

229. Air Chambers. The reversals of direction and the changes in speed of the reciprocating parts of a pumping engine may result in fluctuations of pressure in the discharge or suction piping of such magnitude as to burst the pipe or to cause difficulties in the distribution system or to cause the pump to lose suction. The violence of the pressure fluctuations can be diminished by attaching air chambers to the suction and discharge pipes.

The capacity of the air chamber is based upon the piston displacement. For slow-moving pumps under moderate pressures the capacity of each air chamber should be about equal to one piston displacement. The chamber should be enlarged for high-speed and high-pressure pumps. Air chambers with capacities six to eight times the piston displacement are not uncommon. The air chambers are most effective when they are connected as closely as possible to the water cylinder of the pump, as shown in Figs. 100, 101, and 130. They should be placed on the dead end of a T or on top of the suction chamber, preferably on the opposite side of the chamber from the suction or discharge pipe, and in such a position that rising air will accumulate in them The air chambers should be provided with a glass gage and air valves in order that the volume of air in them may be observed and controlled at all times. The water opening into the air chamber should be of the same size as the pipe in which pressure relief is required.

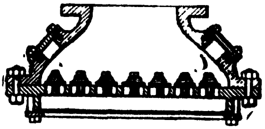
FIG. 107.—Foot valve.

230. Setting of Reciprocating Pumps. A pumping engine should be securely fastened to a firm foundation to prevent shocks and vibrations from being transmitted to the suction and discharge piping. A common and satisfactory method is to bolt the pump to a heavy concrete foundation. A wooden foundation is unsatisfactory, as the pump will work loose, thus throwing undue stresses upon the pipe connections, which may crack them or cause leaks.

The size, length, and arrangement of the suction pipe materially affect the success of the operation of the pump, as stated in Sec. 212. A velocity of not more than 2 f.p.s. in the suction pipe should give a satisfactory size. In the discharge pipe, 4 to 5 f.p.s. will be satisfactory. Air chambers should be used as described in Sec. 229, and foot valves should be used as described in Sec. 228.

The priming of the pump should be provided for, where necessary. A foot valve is essential in almost every case where priming is required. A pump is primed by connecting a high-pressure water line to it so that water can be admitted to the pump from this line; or where the pump discharges directly into pressure pipes to which other pumps are connected, it is simplest to prime by admitting water to it through its own discharge pipe, the water entering through a by-pass around the check valve on the discharge pipe.

A check valve should be placed on the discharge pipe close to the pump in order to relieve the pump valves from the discharge water load when starting and as a factor of safety in the event of accident. A gate valve should be placed on the discharge line beyond the check valve so that necessary repairs to the pump and check valve can be made when the gate is closed. A safety device should be installed to prevent the closing of the gate valve while the pump is in operation. Otherwise the pump may be wrecked.

231. Steam Fire Pumps. The National Board of Fire Underwriters has standardized specifications for pumps intended for fire protection only, as, for example, pumps that are used in industrial plants, buildings, and similar locations. The specifications are not applicable, in all their details, to waterworks pumps. Where fire pumps alone are to be installed, they should be designed in accordance with these specifications.

CHAPTER XIV

ELECTRICAL EQUIPMENT

232. Electric Power. Reliabilty, simplicity, availability, cleanliness, and relatively low costs combine to increase the utilization of electric power in waterworks pumping stations. Electric power is used almost exclusively in smaller pumping stations recently designed. Steam power is to be found in few new installations other than in combined water, light, and power plants and in large pumping stations where economy or other considerations dictate its adoption. Electric current can usually be purchased more cheaply than it can be generated in any but the largest waterworks pumping stations. The universal use of electric power in waterworks practice, as well as in most other fields of engineering, requires a working knowledge of electrical equipment and its limitations. The waterworks designer must know what can be done with electricity, what equipment is available, how to specify the equipment to be installed, how to install it, and the conditions of operation that may affect the type of equipment to be selected. Comprehensive standards of electrical equipment and methods of installation, approved by the National Board of Fire Underwriters and by the American Standards Association, were published in 1940 with subsequent supplements. It should serve as a guide in the design, purchase, and installation of electrical equipment.

233. Electrical Equipment. In the design of a waterworks pumping station in which electric power is to be used, provision must be made for receiving, conveying, and utilizing the current. This involves such equipment as transformers, switchboards, motors, circuit breakers, relays, and safety devices. Knowledge of the nature of electric circuits is necessary in the application of such equipment.

234. Electric Current. The type of electrical equipment used must be suited to the type of electric current available. Very little electric power equipment is interchangeable between types of electric current, and the pumping-station design is commonly limited to the kind of current available on the wires of the public-service company. Most power lines in the United States carry three-phase 60-cycle alternating current. In some communities direct current and other combinations of phase and frequency in alternating currents are available such as single-phase, two-phase, 25-cycle, and 50-cycle currents. Advantages of alternating current include lower first cost and lower maintenance cost of induction equipment as compared with direct current equipment, and the ability to step the voltage up or down by the use of transformers.

Direct current is more costly in transmission and application because of the larger conductors needed and because of the greater first cost and maintenance cost of d-c motors. Direct-current wiring may be more simple than the wiring for polyphase alternating current; motors, batteries, and electrolytic equipment can be on the same circuit; the use of direct current allows better control of motor speeds; there is unit power factor under all conditions; and in certain applications, such as in electromagnetic devices, in starting synchronous motors, and in battery charging, its use may be either desirable or necessary. Where only alternating current is available, direct current can be produced in the pumping station by motor generators or current rectifiers.

In general, polyphase alternating current is used almost exclusively in waterworks pumping stations because of the availability of simple, rugged, low-cost, constant-speed polyphase motors. Such motors can be operated at higher voltages than if direct current is used, thus decreasing the cost of the machine. Where electric current is generated within the pumping station and there are few motors or where the motors are to be operated at variable speed, the use of direct current may be advantageous.

235. Voltages. The voltages used in the transmission of electric power may be much higher than the voltage at which power is delivered in the pumping station. In general, it may be expected that the highest voltage coming into a pumping station will not exceed 13,800 volts; usually it is not over 2,200 volts. Such voltage can be stepped down by transformers to the standards for various applications shown in Table 47.

TABLE 47.—STANDARD[1] VOLTAGES USED FOR PUMPING EQUIPMENT IN WATERWORKS PUMPING STATIONS

D.-c. voltage	Uses	A.-c. voltage	Uses
110	Lighting, motors	110	Lighting, small motors, small appliances
220	Motors-all other purposes	110–120	Lighting, small appliances
		220	Motors, electric ranges
550	Motors-all other purposes	440	Motors
		550	Motors
		2,200	Motors larger than 60–100 hp.
		4,000	
		6,600	Distribution lines and, sometimes, large motors
		11,000	
		13,200	

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[1] A.S.A., National Electrical Manufacturing Association, and A.I.E.E. Standards.

Conditions affecting the voltage to be used include first cost of equipment, cost of maintenance, and danger to personnel. The higher the voltage the lower the cost of copper but the greater the cost of insulation, and the greater the hazard to personnel. A 100-hp. 110-volt motor will cost more than a 100-hp. 550-volt motor. In general, no special hazard exists and no objections are offered by personnel when the voltages are 440 or less. At and below this voltage the cost of the equipment and its installation will usually control in the selection of the voltages to be used in the pumping station.

236. Frequencies. That frequency should be selected which is most suitable for all purposes in the pumping station because the use of different frequencies on different equipment is impractical. Sixty cycles has been found most satisfactory because it avoids noticeable flickering of lamps, which occurs at 25 cycles, and most motors and other induction equipment can be manufactured at less cost at the higher frequency. However, on the basis of power utilization alone, particularly on slow-speed induction motors the lower frequency is advantageous because of less inductive effect, greater simplicity in design, and the better power factor obtained.

237. Phases and Circuit. The number of phases and the type of circuit to be used are of importance in power transmission. They are, therefore, important in power utilization because equipment must be suited to the type of current utilized. Systems of transmission in use include those listed in Table 48, which includes also a summary of the relative advantages of the various circuits. More than three phases are not commonly transmitted on public-service power lines. Single-phase two-wire, single-phase three-wire, and three-phase three-wire circuits are in common use. The change of motors, and some other equipment, from single phase to three phase involves rewinding. However, it is possible, by the use of transformers to interchange between two-phase and three-phase currents, and between three-wire and four-wire circuits.

238. Characteristics of Electric Motors. In the selection of an electric motor conditions to be considered, in addition to the type of current and of circuit available, include (1) first cost and maintenance cost; (2) suitability for the character of work to be done; (3) speed, speed regulation, and speed control; (4) ease in starting; (5) starting current in terms of full-load current; (6) starting torque in terms of full-load torque; (7) power factor; (8) rated power; (9) partial load characteristics; and (10) whether enclosed, semi enclosed, or open. Some characteristics of the more common types of electric motors are listed in Table 49. Motors with other characteristics may be manufactured by special windings, usually at a higher cost than for standard-type motors.

TABLE 48.—RELATIVE ADVANTAGES OF CIRCUITS AND PHASES OF ELECTRICAL SYSTEMS

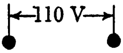
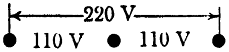
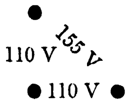
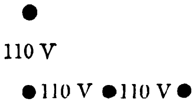
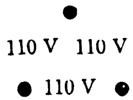
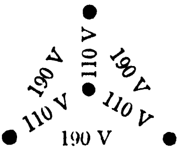
System	Relative weight of copper, based on carrying capacity	Remarks
Single-phase alternating-current or two-wire direct current  	115	Most simple of all forms
Single-phase alternating-current or three-wire direct-current  	All wires same size 87 Neutral 0 7 outside wire 77 5	Allows use of two voltages
Two-phase alternating current three-wire  	Common wire $\sqrt{2} \times$ outside wire 97 5	More simple than four-wire circuit
Two-phase alternating current four-wire  	115	Requires only two transformers, and two phases are insulated from each other, independent in operation
Three-phase alternating current three-wire  	100	Voltages between lines are equal and all wires are same size. Simple compared with four-wire circuit
Three-phase alternating current four-wire  	All wires same size 77 Neutral 0.7 × outside wires: 71.5	Principal savings in copper

TABLE 49.—ELECTRIC-MOTOR CHARACTERISTICS

Type of motor	Characteristics, advantages, and applications
Shunt wound, direct current	*Starting current:*[1] normally about 1.5 times full load *Starting torque:* normally about 1.5 times full-load torque. *Maximum torque:* twice full-load torque. *Speed regulation:* about 10 per cent variation between no load and full load. *Speed control:* adjustable by means of rheostat, varies not more than about 10 per cent for any adjustment. *Sizes available:* up to 200 hp. *Suitable for* frequent starting and stopping with light or medium starting loads, shop machinery, conveyors, fans, centrifugal pumps.
Series wound, direct current	*Starting current:* same as for shunt-wound motor. *Starting torque:* 4.5 times full load but normally 2.25 to 3.0 times full load torque. *Maximum torque:* four times full-load torque. *Speed regulation:* poor, increases dangerously with decrease of load, may "run away." *Speed control* is same as shunt-wound motor. Sizes available from 3 to 200 hp. *Suitable for* load that cannot be removed from motor such as cranes, fans, and railways, and for frequent stopping and starting. Not suitable for centrifugal pumps.
Compound-wound direct current	Characteristics lie between shunt wound and series wound and depend on the character of the windings.
Induction alternating current, polyphase squirrel cage	*Starting current:* five to ten times full-load current. For sizes above about 5 hp. requires special starting devices. Where frequent starting is necessary, sizes below 5 hp. may require special starting windings, condensers, or other starting aids. *Starting torque:* low in terms of full load *Maximum torque:* 2 to 2.5 times running torque. *Speed regulation:* good, may diminish 5 per cent between start and full load. *Speed control:* none, runs only at designed speed. Sizes available 0.1 to 400 hp. *Suitable for* constant load and continuous operation, excellent for fans and centrifugal pumps.
Repulsion-induction, single phase, alternating current	*Starting current:* 1.75 to 2.5 times full load. *Starting torque:* three to five times full load. *Speed regulation and control* at no load is above synchronism; at full load it is similar to that of an induction motor. *Sizes available:* ⅛ to 15 hp. *Suitable for* heavy starting loads or frequent starting and stopping.
Synchronous, alternating current	*Starting current:* four to eight times full load. Hard to start. Field must be excited with direct current. *Starting torque:* very low. *Maximum torque:* 1.5 to 1.75 running torque. *Speed regulation:* perfect, no variation in speed under varying load. *Speed control:* none. *Sizes* available: 20 to 5,000 hp. *Suitable for* steady load and constant operation. Excellent for pumps. Improves power factor.

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[1] Starting currents in this list are stated as 75 to 100 per cent of the blocked rotor values. The free rotor value may be taken as 75 per cent of the blocked rotor value.

239. Induction Motors. An induction motor is an a-c motor in which the armature current is introduced into the armature windings by electromagnetic induction instead of by brushes. Induction motors consist of two concentric masses of laminated iron in the form of short hollow cylinders, of which the outer, called the "stator," is fixed, and the inner, called the "rotor," revolves. The outer surface of the rotor and the inner surface of the stator are slotted or perforated to receive the primary or secondary windings.

There are two types of induction motors: the wound rotor and the squirrel cage. Each receives its name from the construction of the rotor and the stator. Single-phase and polyphase motors resemble each other structurally, but the single-phase is heavier, more costly, less efficient, and more difficult to start and has a lower power factor. A slip-ring wound-rotor motor is an induction motor with the rotor winding brought out to slip rings so that resistance can be introduced into the rotor circuit. The wound-rotor motor is sometimes known as the phase-wound or slip-ring motor.

240. Squirrel-cage Motors. The characteristics shown in Table 49 for the squirrel-cage induction motor make it the most suitable and the most widely used type of motor for driving waterworks pumps. Its low starting torque and high starting current do not prevent its use for centrifugal pumps because the pump may be started with the discharge valve closed to minimize the starting load. Special forms of windings may be used to improve the starting conditions and provide motors suitable for any pumping load.

Ruggedness of construction, simplicity of design, low first cost, low maintenance cost, and other desirable 'features combine to make the squirrel-cage motor suitable for a wide variety of applications. These motors can be modified to provide high starting torque and rapid acceleration, which makes them available for frequent starting and stopping. The first cost may be increased, however, the efficiency lowered, and speed variation during operation increased by such modification. The provision of a flywheel to equalize momentary fluctuations of load may make possible the use of squirrel-cage motors under conditions not otherwise suitable. Another method of increasing the starting torque is to provide a double squirrel-cage winding. This will give a starting torque 2 to 2.5 times full-load torque with a slip of less than 5 per cent.

The efficiency of induction motors at full load ranges from about 85 per cent for small motors to 92 per cent for large machines. The power factor is relatively low, particularly when starting, and is always lagging.

241. Phase-wound, or Slip-ring, Motors. This type of motor is especially adapted for large starting torque and hand control of speed.

The speed is controlled through a rheostat which introduces resistance into the secondary windings in much the same manner as the starting rheostat. These motors are used in sizes above 50 hp. where a high starting torque is required. Wound-rotor motors using starting resistance have a power factor at start of 0.60 to 0.65.

242. Synchronous Motors. A synchronous motor is an a-c generator running as a motor with its armature supplied with alternating current and its field excited with direct current. These motors require frequent attention and must be carefully selected and operated to assure adequate starting, pull-in torque, and efficient operating conditions. Direct current must be available for starting, unless specially wound for alternating current. They have satisfactory operating characteristics for continuous operation under steady load, particularly in large installations. Their outstanding advantage is their improvement of the power factor which can be brought to unity on any line by adjusting the synchronous motor to lag or lead the necessary amount.

The cost of a synchronous motor, its d-c exciter, and its control may be expected to be greater than the cost of a squirrel-cage motor of equal power; but where operating conditions are suitable, the higher efficiency, the leading power factor, and the more attractive rates for power sometimes offered by power companies make the installation of a synchronous motor desirable.

243. Starting Alternating-current Motors. It is desirable, but not always possible, to start a motor under full line current in order to avoid the need for special starting equipment or motor winding. It is usually safe to start induction motors of less than 5 hp. by connecting the full line operating voltage to the motor. Although the momentary current may be large, the motor quickly accelerates, if not too heavily loaded, thus reducing the current passing through the windings before heating occurs. Reduced voltage required in starting induction motors may be obtained by placing adjustable resistances in the starting circuit, thus by-passing the operating fuse or protecting relay. As such arrangements require a large line current, an autotransformer, or "autostarter," arranged for automatic or manual starting may be used. For smaller full-voltage-starting induction motors, the simple magnetic starter, with overload and undervoltage protection, operated by a push button, is widely used.

Wound-rotor motors are protected in starting by inserting resistance into the rotor winding. A thermal relay, or fuse, and a line switch protect the stator current, the starting circuit being protected with fuses permitting a current of 50 per cent of full-load current. As the motor gains speed the starting resistance is reduced and finally cut out.

The starting characteristics of synchronous motors are varied to suit the load, but under any conditions the starting of a synchronous motor is difficult. For this reason frequent starts and stops are undesirable. Such motors are usually made self-starting by providing the rotor with an induction winding which permits the starting of the motor as an induction motor with the starting torque adapted to the load. Under such conditions, however, the motor draws a heavier starting current than an induction motor of the same rating, so that the application of other expedients may be necessary to get the motor started.

The line current required to start a centrifugal pump connected to an electric motor can be minimized by starting with the pump discharge valve closed. Under such conditions the torque at full speed will be about two-thirds of that under full load when the valve has been opened.

Starting switches and push-button relays may be assembled on a switchboard or placed separately near the motor. Under the latter arrangement the starting switch and fuses should be enclosed in a metal box, the equipment being called a safety switch.

244. Motor Speeds. High-speed motors cost less than slow-speed machines because more power is delivered per unit weight of machine. The limit of speed is fixed by the strength and durability of the parts and by the possibility of applying the speed to the load. Slow-speed motors are used for machines that revolve slowly and that cannot be connected through belts or reduction gears. Most motors operate with the highest efficiency at their rated speed. This is not true of series-wound motors, which operate equally well at almost any speed. Only a limited number of speeds is available in standard induction motors. On 60-cycle current synchronous speeds available are 1,800 and 1,200 r.p.m.

245. Variable Speeds. Where an electric motor is used to drive a centrifugal pump, some method of varying the speed of the pump is desirable, for economical operation, unless the load on the pump is constant. Other applications of electric motors may require speed control. This may be obtained through varying the speed of the motor, or through mechanical devices in the motor or between the motor and the pump.

246. Speed Control. Speed control for d-c motors is usually secured by the use of a rheostat connected in either the field or the armature circuit, or one in each circuit. Multiple-voltage speed control is usually secured through the use of two voltages on a three-wire system, thus making possible a slow speed that bears the same relation to the high speed as the lower voltage bears to the higher voltage on the three-wire circuit. Intermediate speeds are secured through the use of field or armature rheostats.

In general, the ordinary squirrel-cage induction motor is not suitable for close speed control giving uniform gradations of speed between limits. Where a few definite speeds will be satisfactory a so-called "multispeed motor," running at two, three, or four different speeds, may be used.

Wound-rotor motors should be used where close speed control is desired on polyphase alternating current. The speed of these motors can be controlled by placing variable resistance in the rotor circuit. Difficulties with speed control of wound-rotor motors are low efficiencies and relatively poor speed regulation which affects the speed with change in load.

Polyphase induction motors of the commutator brush-shifting type permit uniform gradation of speed from 40 per cent above to 60 per cent below synchronous speed. The polyphase induction motor, with commutator and brushes, will give any speed from 0 to 50 per cent above synchronous without the introduction of resistance losses.

A gear motor is any type of electric motor which has built into it a speed-reduction gear that will give various shaft speeds. Gear motors are built to give speeds from 2 to 1,550 r.p.m.

In general, the highest efficiency of an a-c motor is obtained when the speed is nearest synchronism, *i.e.*, with the least slip.

247. Speed-reducing Devices. Since the speed of most electric motors is too high for direct connection to centrifugal pumps, some form of speed-reduction device, other than variable-speed motors, may be used. Among these devices may be included (1) gear trains, (2) belts and pulleys, (3) hydraulic couplings, and (4) mechanical couplings.

248. Special-duty Motors. There are many different types of motors, each of which is adapted to some special duty or operating conditions. Among those suitable for special conditions in waterworks may be included (1) dripproof, (2) splashproof, (3) waterproof, (4) submersible, (5) gear, and (6) brake. A dripproof motor is encased so that drops of water falling on the motor at an angle less than 15 deg. from the vertical cannot enter the motor directly or by running along a horizontal or inclined surface. Similarly, a splashproof motor is safe against water approaching it at an angle less than 100 deg. from the vertical. A waterproof motor is enclosed so that a stream of water from a hose, 1 in. or larger in diameter under a head of 35 ft. and from a distance of 10 ft., can be played on the motor without leakage except possibly along the shaft, with provision for preventing this water from entering the oil reservoir and for automatically draining the motor. Waterworks pumping stations offer many conditions where such motors are needed.

Since the enclosing of a motor limits its ventilation and cooling, open motors should be used where possible, as their ratings are higher and their costs comparatively lower. For similar reasons motors should be installed in cool well-ventilated surroundings.

Submersible motors can be operated when submerged in water. Their use is infrequent. Gear motors, as mentioned on page 221, are variable-speed machines. Brake motors have a brake built into them which brings the motor, or motor and load, rapidly to a stop when the current is cut off.

249. Rating of Electrical Equipment. Electrical equipment, particularly motors, are rated in accordance with the power delivered with a limiting temperature rise under standardized time and conditions. The permissible temperature rise is in the neighborhood of 40 to 50°C., dependent on the construction of the motor or equipment. Since time is required to raise the temperature, it is possible for electrical equipment, particularly motors, to carry an appreciable overload for a short period of time, the greatest hazard under such conditions being sparking, or internal short-circuiting. Standard ratings for electrical equipment have been adopted and published by the American Institute of Electrical Engineers. These standards enter into detail concerning the requisite performance of electrical equipment and should be consulted by an engineer responsible for the selection of electrical equipment.

Where motors are to be operated in ambient temperatures above 40°C. the construction of the motor, particularly its internal insulation, must be adapted to the higher temperatures, or the load on the motor must be reduced below the standard rating.

250. Operating Hazards for Polyphase Induction Motors. Hazards to which polyphase induction motors are subjected in operation may be classified as those due to (1) variations in characteristics of electric power delivered to the motor, (2) improper operation, and (3) improper care.

The current delivered to the motor may vary in phase, in frequency, and in voltage. If one or more phases are interrupted while the motor is idle, the motor will not start; but if it has been running at the time of the interruption or if it is started by the application of outside power, as by pulling on the belt, the motor will run at normal speed as a single-phase motor and will deliver about 60 per cent of normal-load torque, with a pull-out torque about two-thirds to one-half normal. The effect of operating a polyphase motor under full load and single-phase conditions will be to overload the motor and to lead to its probable destruction. The interruption of one or more phases of an a-c circuit is not unusual and must be guarded against in motor operation.

Voltage affects the torque of an induction motor. Both the starting and operating torque vary closely as the square of the voltage. Hence a drop in voltage may cause a motor to refuse to start, or to pull-out if in operation. A motor thus stopped with current in its windings is open to destruction.

If a motor is operated above normal voltage, the core losses are increased approximately as the square of the voltage and the copper losses are decreased. An excessive increase in voltage endangers the insulation of the windings as a result of the rise in temperature of the core. The slip of induction motors varies approximately inversely as the voltage.

The speed and torque of induction motors vary approximately as the frequency of the current. At normal operating load and with increased frequency the core losses are increased, the result being increased heating despite the lower speed. Since the horsepower required by a centrifugal pump varies as the cube of the speed, a decrease in frequency might quickly result in overloading the motor.

In general, induction motors will not be seriously affected by variations of 10 per cent, more or less, in either voltage or frequency. Interruption of phase is, however, a serious hazard. The sudden interruption of power will have no serious effect on an induction motor unless it is connected to a reversible load, such as a centrifugal pump. Under such circumstances both the motor and the pump must be protected against reversal. This may be accomplished by the installation of check valves on the pump discharge line or other devices mentioned on page 213. Unless a motor is equipped for automatic self-starting under load, it must be protected against the resumption of power in the event of its interruption.

251. Protection of Motors and Operators. Electric motors should be protected against the hazards of over- and undervoltage, over- and underload, reverse current, reverse power, reverse phase, lightning, and other current surges. Protection against hazards are provided mainly by devices that break or ground the electric circuit when danger threatens.

Fuses, depending on the melting of a strip of metal carrying the current, are used to protect motors against overload. They are limited in capacity to 600 amp. Time-lag fuses are designed particularly for the protection of small motors. They are used to permit the momentary surge of current up to 500 per cent of rating, in starting the motor, but to interrupt the circuit if the temperature of the motor rises excessively. The thermal cutout is designed and functions similarly to a plug fuse and is used to protect small motors against overloads. They protect the motor against overload, whereas the time-lag fuse protects the motor against too high a temperature from any cause. Fuses and thermal cutouts do not protect against lightning or surges of current.

Circuit breakers, actuated either thermally or magnetically, are available to interrupt currents from 15 amp. up, under all conditions of service and, if required, to close the circuit automatically when the current has returned to normal. Circuit breakers, actuated by magnetic devices or relays, are available for protection against all the hazards listed for motors, except to arrest lightning.

The lightning arrester is a device placed on an electric circuit to ground excess voltage in the circuit and to restore the circuit on the return to normal voltage limits. Since the energy grounded through a lightning arrester may be high, the arrester should be placed in a nonhazardous location away from other equipment or combustible materials, or out of doors.

Motor frames may become charged by leakage of current due to breakdown of insulation, by stray currents, or induced current, all of which endanger the motor and the operator. This hazard can be eliminated by grounding all motor frames and carefully inspecting and maintaining this ground. Probably the most satisfactory ground is a connection to a water-supply pipe. The practice of making such grounds is widespread and is generally permitted.

252. Setting of Motors. A foundation that is large enough on which to set the motor and is sufficiently massive to absorb vibrations of the motor without transmitting them to its surroundings, and will hold the motor in alignment under all the loads for which it is designed, is required for a motor setting. Concrete, brick, or a combination of these two and, sometimes, wood or metal are used in satistactory foundations, but no definite ratio of motor mass or speed to foundation mass can be set. Experience, based on a study of successful settings, must guide. Some typical motor foundations are illustrated in Figs. 97 and 108. Dimensions of motors may be approximated by a study of catalogues, or they may be obtained on request from manufacturers.

The designing and construction engineers are responsible for creating an adequate and level foundation. The setting and alignment of the motor require the attention of a specialist, particularly for most forms of power drives other than belt or chain.

253. Transformers. It may be necessary to provide space for transformers in an electrically equipped pumping station, usually to step down the voltage from that carried on the transmission lines to the voltage to be used by the equipment in the pumping station. Because of the hazards of fire and high-voltage currents, transformers should be placed in a fireproof room or in a location protected from traffic in the pumping station. Rules for the safe installation of transformers are included in the National Electric Code.[1] Weights and dimensions of transformers for various purposes are listed in handbooks and in manufacturers' catalogues.

254. Relays and Circuit Breakers. Relays may be used for a multitude of services, but in an electrically equipped pumping station their principal use is to actuate circuit breakers in the protection of equipment and to start and stop motors, pumps, and other equipment Relays are available that will actuate a circuit breaker under almost any condition such as change in voltage, phase, or cycle of current; overload, speed, or temperature of motor; or filling or emptying of a reservoir. Two types frequently used are the instantaneous or dashpot, and the induction The latter is the most commonly used.

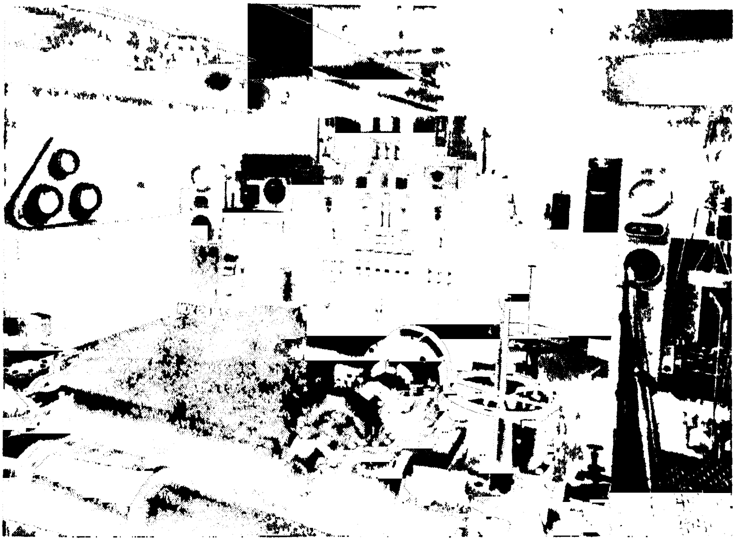

FIG 108.—Interior of Towson automatic pumping station, Baltimore, Md. Showing motor-driven centrifugal pumps, meters, and other appurtenances (*Courtesy of General Electric Company*)

255. Switchboards. A switchboard is an essential part of the equipment of a pumping station or purification works using electrical equipment. The equipment placed on the switchboard includes starting devices, relays, meters, and recording and indicating devices. The equipment can be neatly arranged on the board with devices controlling separate circuits arranged on separate panels. A circuit breaker should be installed on the incoming line from the power plant, which will open all short circuits on the system. There should be smaller circuit breakers, with thermal-overload protection on each of the motor circuits. Knife switches should be used for direct current only with voltages below 250, and for alternating current with voltages below 500. At higher voltages oil or air-gap switches, controlled by push-button relays, should be used.

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[1] See ABBOTT, A L, "National Electrical Code Handbook," 5th ed , p 201, McGraw-Hill Book Company, Inc , New York, 1940

Types and space requirements of switchboards are listed in handbooks and catalogues. Rules for their installation are to be found in the National Electric Code.[1]

256. Installation of Electrical Equipment. Standard practice should be followed in the installation of electrical equipment. This constitutes the use of adequate-size wire, supports, protection, and equipment for the loads and conditions contemplated. Details of this practice have been codified and published in the National Electric Code for Electric Wiring and Apparatus. A copy should be in the hands of every designer, purchaser, and operator of electrical equipment.

257. Automatic Pumping Stations. Electrical equipment lends itself favorably to automatic operation and remote control of pumping stations, resulting in reliable, efficient, and economical operation.[2] Automatic stations are those in which the equipment is started and stopped through relays which are controlled by reservoir levels or system pressures or by faults in, or failure of, station equipment or connections. Remote-control stations are those in which all, or the major part, of the equipment in the pumping station is controlled by operators in a remote location. Such stations may be designed to operate for days at a time without attendance, reports of the conditions of station operation being telephoned, on demand, by robots. Automatic stations make possible the exercise of ingenuity in waterworks design, resulting in economies of construction and of operation.

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[1] *Ibid*, p. 32.

[2] CANARIIS, S A, "Modern Methods of Controlling Electrically Operated Pumping Stations," *Jour. A.W.W.A.*, May, 1936, p 583.

CHAPTER XV

CENTRIFUGAL PUMPS

258. Description. A centrifugal pump is a device that raises a liquid by centrifugal force created by a wheel, called an impeller, revolving in a tight casing. In operation the liquid enters the pump at the center of the impeller, called the "eye." It enters in a direction normal to the plane in which the impeller revolves and is thrown to the periphery by the centrifugal force resulting from the revolution of the impeller. The fluid passes through the channel between the rim of the impeller and the casing and issues at the discharge under pressure. Sections through two types of centrifugal pumps are shown in Figs. 109 and 110.

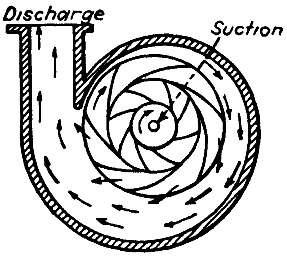

Fig. 109.—Centrifugal pump with turbine diffuser vanes.

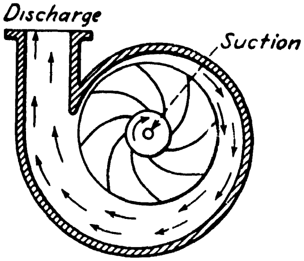

Fig. 110. Volute pump

259. Status and Performance. In waterworks practice centrifugal pumps are used almost exclusively in all but the largest pumping stations, in a few of which triple-expansion reciprocating pumping engines are to be found. Centrifugal pumps are available commercially for almost any capacity that might normally be required, and for lifts up to 700 ft. in one stage.[1] There is no theoretical limit to the number of stages that can be placed in series to increase the lift. Customary maximum lifts, per stage, are nearer 250 ft., however. Efficiencies up to 93 per cent have been attained by large pumps,[2] but, in general, the efficiencies of small pumps are lower. Speeds of 900, 1,200, 1,800 r.p.m., and even higher, are used for waterworks pumps. Small pumps are now being manufactured with speeds up to 6,000 r.p.m.[1]

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[1] See also Burgess, John, "Centrifugal Pump Performance," *Jour. A.W.W.A.*, May, 1941, p 837.

[2] *Ibid*, and R W. Angus, "Centrifugal Pump Development in 25 Years," *Eng. and Contract Record*, March, 1945, p. 72.

Advantages of the centrifugal pump, particularly when compared with other types of pumps, include (1) relatively small space for any given capacity; (2) rotary rather than reciprocating motion; (3) adaptability to high-speed driving mechanisms, such as electric motors, steam turbines, and gas engines; (4) low first cost; (5) simple mechanism; (6) simple operation and repair; and (7) safety against damage from high pressure because of limited maximum pressure that can be developed. The pump is thus safe against damage if the discharge valve is suddenly closed, although in a few low-lift pumps with special characteristics the motor might be overloaded under these conditions. Disadvantages include the following: (1) Rate of flow cannot be efficiently regulated for wide ranges of performance. (2) Speed cannot always be adjusted to the prime mover without the use of speed-reducing or speed-increasing gear. (3) Pumps are not self-priming without additional mechanism. (4) Suction lift may be restricted. (5) Air leaks on the suction side easily affect pump performance. (6) If stopped with the discharge valve open, the pump may run backward. (7) In general, high efficiency is maintained over only a narrow range of head and discharge.

260. Cost. The first cost of a centrifugal pump is, ordinarily, less than the first cost of other types of pumps of the same capacity and lift. Among the factors affecting first cost are included the discharge pressure, speed, number of impellers in series, quality of material and construction, market conditions, and capacity. In general, it may be said that first cost varies with capacity, other factors being equal.

The relative economy of centrifugal pumps should be compared with that of other pumps on a capitalized cost basis, including first cost, installation, space requirements, mechanical efficiency, lubrication, and other maintenance costs, and durability or life. Each of these items, except possibly that of mechanical efficiency, will usually be found favorable to the centrifugal pump. Since the efficiency of the pump may greatly affect its economy, careful attention should be paid to this factor in the selection of a pump.

261. Theory. The general physical laws applicable to a centrifugal pump can be expressed as

$$F = Ma = \frac{MV^2}{r} = \frac{WV^2}{gr} = \frac{W\pi^2N^2r}{900g}$$

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[1] Burgess, *loc. cit.*

where F = centrifugal force
a = radial acceleration
g = acceleration due to gravity
M = mass of moving body
N = speed, r.p.m.
r = radius of path
W = weight of moving body

The theoretical height to which a column of water will rise under the effect of centrifugal force produced by a revolving impeller is

$$h = \frac{V^2}{2g}$$

where V is the linear velocity of the tip of the impeller. V^2 is equal to $r^2\omega^2$, in which r is the radius of the impeller and ω is its angular velocity.

The equation of the surface curve of a body of water made to revolve in a cylindrical vessel, as shown in Fig. 111, is, therefore,

$$y = \frac{\omega^2 x^2}{2g} \tag{1}$$

This is known as the equation of a forced vortex. It is the shape of the pressure curve of the water in the casing of a centrifugal pump when the discharge valve is closed and the impeller is revolving. It shows that the pressure exerted by a centrifugal pump varies as the square of the speed of the impeller. The value y when x is r is known as the pressure of impending delivery. In most pumps, since there is some leakage around the impeller blades into the casing, the pressure of impending delivery is seldom exactly as shown in the expression. It may be either higher or lower, depending upon the shape of the impeller blades, the rate of leakage, and other factors. It will usually be found that the actual pressure of impending delivery is within 10 to 15 per cent of the value of this expression.

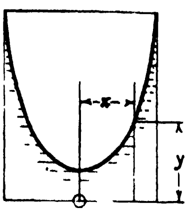

Fig. 111 A forced vortex.

If the impeller of a centrifugal pump is revolved while the discharge valve is closed and an orifice is opened at any point in the casing, water will flow from the orifice under a head of $\omega^2 x^2/2g$. Since the rate of discharge through an orifice is

$$Q = kA\sqrt{2gh}$$

where A is the area of the orifice, and h, the head on the orifice, is equal to $\omega^2 x^2/2g$, it follows, therefore, that

$$Q = kA\omega x \tag{2}$$

Hence, the rate of flow, or the discharge from a centrifugal pump, varies as the speed of revolution of the impeller.

The torque exerted by the impeller of a centrifugal pump is

$$T = \frac{Qwr_pV_w}{g} \tag{3}$$

where T = torque
Q = rate of discharge past impeller blade
w = unit weight of water
r_p = radius of impeller
V_w = tangential velocity of the water at impeller tip

The power exerted by the impeller is equal to the torque multiplied by the angular velocity; therefore,

$$P = T\omega = \frac{Qwr_pV_w\omega}{g} = \frac{QwV_tV_w}{g} \tag{4}$$

in which V_t is the linear velocity of the impeller tip, and other symbols are as in formula (3).

262. Relationships of Characteristics. Relationships between characteristics of centrifugal pumps, as developed from theoretical considerations, other conditions being unchanged, may be summarized as follows:

(1) Q varies directly as the number of impellers in parallel.
(2) H varies directly as the number of impellers in series, *i.e.*, the number of stages.
(3) When N varies, Q varies directly as N; H varies directly as N^2; P, the power, varies as N^3; Q varies as $H^{1/2}$; and P varies as $H^{3/2}$.

These theoretical relationships do not hold exactly, because of errors in the hypotheses of the conditions of flow of water, but they are sufficiently accurate for rough approximations. It is found by test that for impending delivery the ratio $V_t/2gh$ varies between 0.95 and 1.09 and that for rated discharge or maximum efficiency this ratio lies between 0.10 and 0.30. If this ratio is called α and we allow $\beta = V_w/\sqrt{2gh}$, in which V_w is the velocity of the water relative to the impeller, then β is approximately equal to α for rated discharge or maximum efficiency. The important relationship between H and Q is shown by the characteristic of the pump discussed in Sec. 264.

263. Centrifugal Pump Series. Centrifugal pumps when built in different sizes, but of the same proportions in every respect, constitute a series and are said to have homologous impellers. Advantages of series pumps is that knowledge of the characteristics of one pump of a series can be used as a basis of prediction of the performance of another pump of the same series. The outstanding relationships among pumps of the same series are that when N is constant and D is the diameter of impeller

Q varies directly as D^3
H varies directly as D^2
P varies directly as D^5

The specific speed of a pump is the speed at which the impeller of a series would run when discharging 1 g.p.m. under a 1-ft. head. Since the specific speed of all pumps of the same series is the same, then, if the head, discharge, and speed of a pump are known, the head, discharge, and speed of any pump of the same series *under different conditions* can be predicted.

Specific speed can be expressed algebraically as

$$N_s = \frac{\text{r.p.m.}\sqrt{\text{g.p.m.}}}{H^{3/4}} \qquad (5)$$

where N_s = specific speed, r.p.m.
r.p.m. = speed of revolution of pump
g.p.m. = discharge from pump
H = discharge head, ft.

The *type* of an impeller is the ratio of its diameter to its width. It is possible to obtain the same discharge pressures from a series of impellers of different diameters running at different speeds, but for the same discharge the type (width) of the impeller would be different. In order to reduce skin friction, it is desirable to operate the impeller at as slow a peripheral velocity as possible, and impeller types should be designed to permit of slow peripheral speeds.

264. Characteristic Curves. The characteristic curves or, more commonly, the characteristics of a centrifugal pump are the relations between the various conditions affecting its performance. They are usually expressed graphically with the rate of discharge Q as the abscissas and the other factors plotted, at constant speeds, as ordinates, such as the head H, the power P, and the efficiency E. *The characteristic*, at any particular speed, is the graph showing the relation between H and Q, with values of H plotted as ordinates. Other methods of plotting characteristics are also used.

Characteristics at one speed are shown in Fig. 112, and at different speeds in Fig. 113.

A study of the characteristics of a centrifugal pump is necessary to determine the best conditions under which it should be operated. For example, Fig. 112 shows that, at the speed shown, the pump will discharge 800 g.p.m. against no other head than 120 ft. To reduce the discharge by closing the discharge valve will increase the head, the power, and the efficiency. The highest efficiency will occur at a discharge rate of about 600 g.p.m. Figure 113 shows that where it is possible to vary the speed of the pump the range of N, Q, and H for highest efficiency can

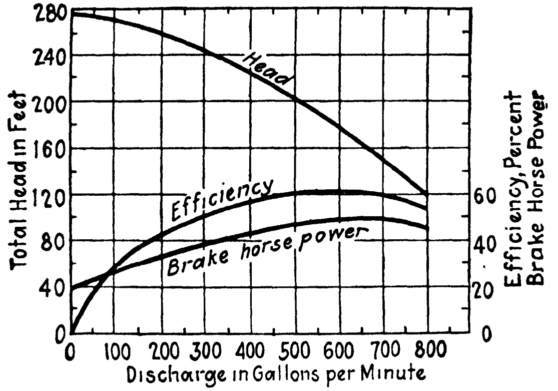

FIG. 112. Characteristics of a centrifugal pump at constant speed. (*From Eng. News-Record, Vol.* 91, *p.* 563.)

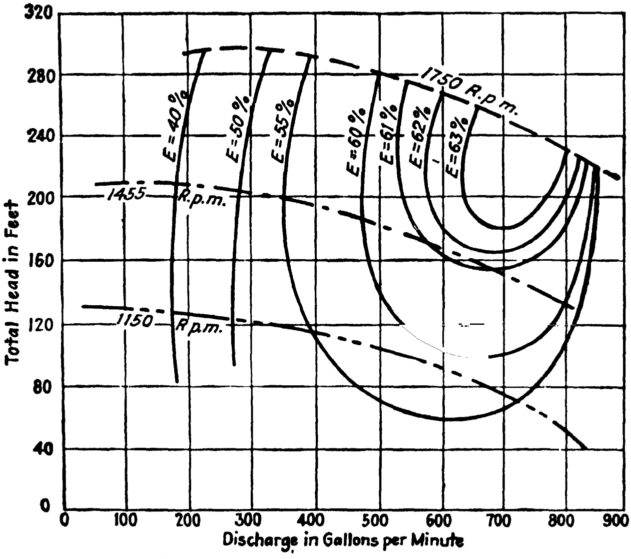

FIG. 113. Characteristics of a centrifugal pump at different speeds. (*From Eng. News-Record, Vol.* 91, *p.* 564.)

be obtained. It shows that this pump should be operated at 1,750 r.p.m., with discharge of about 725 g.p.m. at a head of 255 ft. A pump with flat characteristics at all speeds and a wide plateau at the highest efficiency is most desirable.

Centrifugal pumps[1] should be selected on the basis of their characteristics and operated where the efficiency curve is as flat as possible. Scales to which characteristics are plotted should be scrutinized, as the shape of the curve can be changed materially by changing the scales.

265. Types. Centrifugal pumps are classed as volute or turbine pumps. A diagram of each is shown in Figs. 109 and 110. In a volute pump the channel into which the water flows on leaving the impeller has a volute shape of such proportions that the velocity of flow is the same at all points in the channel. A turbine pump is a centrifugal pump in which the channel into which the water flows on leaving the impeller has the same cross-sectional area throughout. The velocity of water leaving the impeller blades is changed in direction and velocity head is changed to pressure head by means of diffusion vanes.

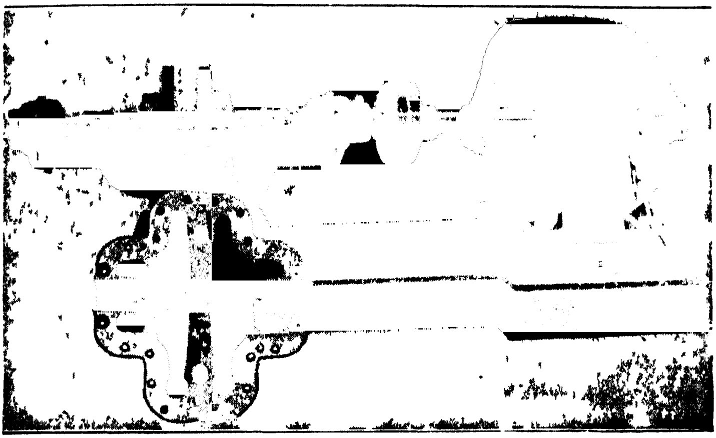

FIG. 114 —Multistage centrifugal pump with horizontally split casing. Note the electric-motor drive and flexible coupling (*Courtesy of American Well Works.*)

Volute pumps are lower than turbine pumps in first cost, and they are more suitable to the handling of liquids containing grit and large particles of suspended matter. Higher efficiencies can be secured from turbine than from volute pumps, but their greater cost and the greater wear on diffusion vanes, combined with other reasons, are diminishing their use in waterworks practice.

Centrifugal pumps also are classed as single stage, two stage, etc., in accordance with the number of stages of pressure developed by the impellers. A multistage centrifugal pump is illustrated in Fig. 114. In the development of discharge pressure it may be inefficient to develop the entire pressure by a single impeller, as either the diameter or the speed, or both, may be undesirably great. Since the discharge pressure from a centrifugal pump is the sum of the initial pressure at the inlet and the pressure energy added by the impeller, increased pressure can be secured by leading the discharge from one impeller into the suction of another. The discharge pressure is directly proportional to the number of stages, other things being equal.

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[1] See also BECKWITH, H. E., "Centrifugal Pumps," *Water Works & Sewerage*, April, 1940, p. 30.

266. Selection of Size of Pump. Centrifugal pumps should be selected to operate at a constant speed under the conditions of head, speed, and discharge that will provide the highest efficiency and lowest cost of power. It will probably be more economical to provide a number of small pumps to operate at highest efficiency and, in addition, to provide a single unit that can be operated at different speeds, or to provide, in addition to the pump or pumps that are to run at constant speed, an equalizing reservoir to care for the fluctuations of demand. Another consideration mitigating against the use of large pumps operating at less than full capacity is the practice, used by some power companies, of basing the power-demand charge on the total capacity of the power units in the pumping station.

The designation of the size of a centrifugal pump is stated by giving the diameter, in inches, of the discharge opening. The capacity in gallons per minute may be roughly determined by multiplying the square of the size by 25. This rule is based on the fact that the velocity of the water through the discharge opening is about 10 f.p.s. This velocity may vary between 5 and 15 f.p.s.

267. Parts. The simplicity of a centrifugal pump is indicated by its parts, which are illustrated in Fig 115. Bronze or gun metal is desirable in parts in contact with water. It is especially desirable that the impeller be of bronze or gun metal in order to resist corrosion. The impeller should be machined and polished to reduce friction, and the sides of the impeller should be enclosed so as to avoid friction between the moving water and the pump casing. It is important that the impeller should be balanced both statically and when in rotation, as an unbalanced impeller may cause heating of the bearings and vibration and noise or the failure of the pump.

Special types of packing rings are illustrated in Fig. 116. They are called labyrinth packing rings and are used between the impeller and the casing to minimize leakage and maintain efficiency. Interior shaft bearings should be of hard bronze. They are placed between impellers and at bottoms of stuffing boxes that are designed for water lubrication only. Lignum-vitae bearings are to be avoided in centrifugal pumps, regardless of lubrication or other conditions. Exterior bearings should be easily accessible. They should be designed for oil lubrication and should be provided with deep oil wells and ring oilers.

The casing of all but the smallest pumps are usually split horizontally along the center, as shown in Fig 114, so that they may be removed to permit inspection and repair of the pump.

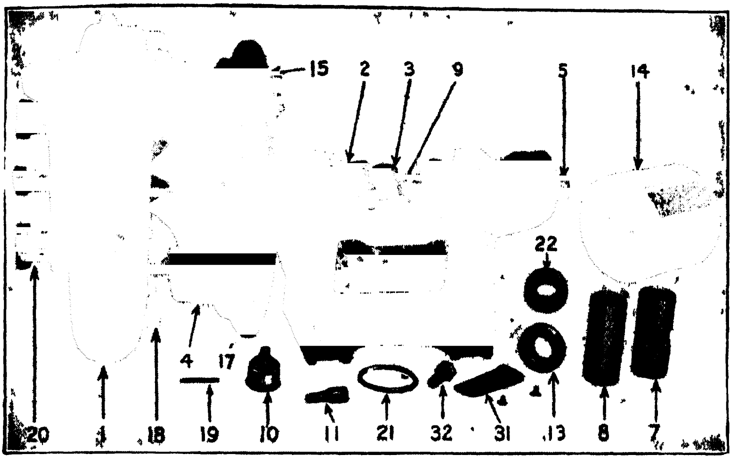
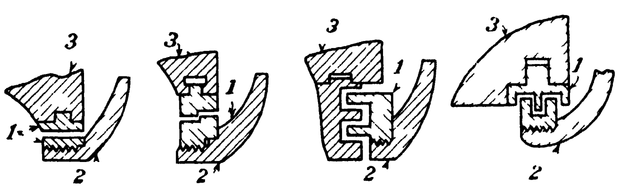


Fig. 115.—Parts of a centrifugal pump (*Courtesy of American Well Works*)

1.	Shell	13	Ball bearing
2	Combination packing box and bearing	14	Pulley
3.	Gland	15	Cover
4	Runner or impeller	17	Shell gasket
5	Shaft	18	Studs and nuts Shell
7.	Bearing sleeve	19	Runner key
8.	Cover sleeve	20	Studs and nuts Suction.
9	Packing-box bolts and nuts	21	Oiling ring
10	Grease cup	22	Thrust collar
11	Grease piping coupling and nipple	31	Oil cover and screws.
12	Packing (not illustrated)	32	Cap screws Cover.



(1)-Packing ring (2)-Impeller (3)-Casing

Fig. 116 Packing rings

Thrust bearings or thrust balancing devices are used to take up the end thrust which occurs in even the best designed pumps To overcome end thrust, pumps are designed with opposed impellers, double suction; or two pumps with their impellers opposed may be placed on the same shaft Because of inequalities in wear, workmanship, or other conditions, end thrust will occur and must be cared for Various types of thrust bearings are in successful use, *e.g.*, the piston disk, roller, and marine types, which are illustrated in Fig. 117. Flexible shaft couplings should be used if the shaft of the driving motor and of the pump are in the same line, as direct alignment is difficult to attain and to maintain. Where connected to steam turbines, reduction gearing and rigid couplings are usually used. Flexible couplings are of various types. One type is shown in Fig. 114. A rigid coupling would be formed by bolting the shaft flanges together.

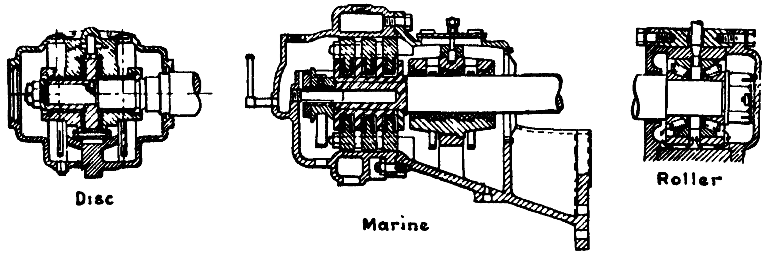


FIG. 117.- Types of thrust bearings. (*From Standards of the Hydraulic Society*)

The use of hydraulic or fluid-filled couplings and of magnetic couplings between electric motors and centrifugal pumps is a comparatively recent practice which overcomes many of the difficulties of the older type friction or flexible couplings. In such couplings[1] there is no solid contact between the power unit and the driven unit, power being transmitted by the friction of a fluid, which occupies an enclosed space connecting the ends of the two shafts, or by means of an induced current.

The stuffing box shown at 2 in Fig. 115 is usually packed with metallic packing, hemp, or asbestos, between which is a lantern gland, in order to permit a small amount of leakage. A drip box is placed below this gland to catch the leakage and to return it to the pump. The leakage aids lubrication, and the tightening of the gland would cause binding of the shaft. The gland on the suction side of the pump should be connected by a small pipe to the discharge chamber in order to keep a constant supply of water for lubrication and to prevent the entrance of air to the suction end of the pump.

268. Setting.[2] Centrifugal pumps are driven by steam engines, steam turbines, internal-combustion engines, and electric motors. Steam turbines are sometimes arranged to drive an electric generator and a centrifugal pump through a speed-reducing gear on the same shaft.

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[1] SINCLAIR, H. "The Transmission of Power by Fluid Couplings," *Engineering*, Apr. 29, 1938, p. 487.

[2] See also "Standards for the Installation of Centrifugal Fire Pumps," National Board of Fire Underwriters, September, 1944.

The electric motor is usually considered to be the most satisfactory driving mechanism. The engine or motor may be connected to the pump by a belt, gearing, or shafting. The simplest and most efficient arrangement is the use of an electric motor and a pump mounted on the same shaft, as illustrated in Fig. 118. If a centrifugal pump and a motor are to be operated on the same shaft, both the pump and the motor should be attached to the same rigid foundation so that the alignment of the shaft will remain true. Because of the difficulty of securing permanent, true alignment, flexible couplings are used on the shaft to allow some variation.

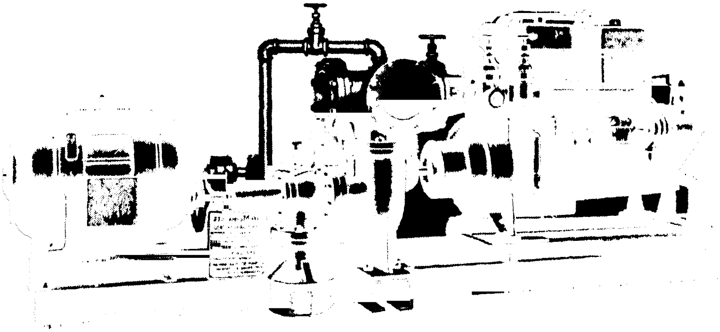

FIG. 118.—Centrifugal fire pump driven by either an electric motor or a gas engine. (*Courtesy of Fairbanks, Morse & Co.*)

Whatever the method of connecting the motor and the pump, it is highly desirable to provide for variable speed of operation in order to obtain the greatest possible efficiency. Hydraulic and magnetic couplings are useful also in allowing control of the pump speed without affecting the motor speed.[1]

Centrifugal pumps are not ordinarily self-priming Hence, in setting and in operation, care must be taken to avoid the loss of suction and to permit its easy restoration. These conditions are best secured by setting the pump below the source of supply so that there is a positive pressure at all times on the suction line. It is undesirable to set the pump with the casing submerged, because of the difficulties of making necessary repairs and adjustments. If the pump is set higher than the level of the water in the suction pit, a foot valve may be required to permit priming of the pump or to prevent the pump from running backward in the event of shutdown with the discharge valve open. The suction pipe should be submerged sufficiently to prevent drawing air into it. This can be done by submerging it about four diameters and keeping the velocity of entrance low. Richardson[1] has given empirical relations between the entrance velocity V, in feet per second into a circular bell, and the minimum submergence S, in feet, which can be formulated approximately as $V^2 - 17S + 10 = 0$. Where no strainer is used, the minimum distance from the face of the bell to the bottom of the reservoir should be $D/2$, where D is the diameter of the suction bell. The distance should be increased where a strainer is used.

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[1] See also PETERSON, A., "Adjusting Centrifugal Pump Speed Cuts Operating Costs," reprint from *Power*, July and September, 1940, with changes; and CAMP, C. W., "The Magnetic Drive," *Water & Sewage Works*, June, 1946, p 243.

The suction line to the pump should be (1) as short and as straight as possible; (2) at least one size larger than the pump suction nozzle; (3) connected by an eccentric reducer, with its top horizontal, to avoid the accumulation of an air pocket; and (4) free from "high points" conducive to the formation of air pockets.

269. Cavitation.[2] It has been found that cavitation may occur in a centrifugal pump as a result of too great a speed or too high a suction lift.[3] The permissible height of suction lift to avoid cavitation is related to the specific speed, as indicated by the data in Table 50 which have been computed from information by the Hydraulic Institute.[4]

It is to be noted from this table that high-speed low-head pumps are restricted to lower suction lifts.

Example: Find the limiting suction lift for a single-suction pump with $N = 2{,}070$ r.p.m., $H = 50$ ft., and $Q = 2{,}000$ g.p.m.

Solution: N_s the specific speed

$$N_s = \frac{2{,}070 \times 44.8}{18.8} = 4{,}930$$

K, from Table 50 $= HN_s^{1.7}$

$$K = 50 \times 4{,}930^{1.7} = 94{,}000{,}000$$

Reading in the table this value of K for a single-suction pump corresponds to a suction lift of zero.

If, however, the same pump is reduced to half the speed, or 1,035 r.p.m., the limiting head can be computed as above. It will be found to be 25 ft.

A noisy pump may indicate cavitation. The admission of a small amount of air into the suction pipe may stop the noise without materially affecting the efficiency of the pump.

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[1] See RICHARDSON, C. A., "Submergence and Spacing of Suction Bells," *Water Works & Sewerage*, June, 1941, p. R-25.

[2] See also WISLICENSUS, G. F., R. M. WATSON, and I. J. KARASSIK, "Cavitation Characteristics of Centrifugal Pumps Described by Similarity Considerations," *Trans. A.S.C.E.*, January, 1939, p. 170.

[3] See also ANGUS, R. W., "Cavitation in Pumps," *Water & Sewage Works*, April, 1946, p. 125.

[4] See also "Specific Speed Curves," *Water & Sewage Works*, April, 1946, p. R-80.

TABLE 50.—PERMISSIBLE SUCTION LIFT TO AVOID CAVITATION IN CENTRIFUGAL PUMPS
(Lift in feet)

Permissible suction lift, ft.	Single-suction, mixed flow, $K = HN_s^{1.33}$			Single suction, with shaft through impeller eye, $K = HN_s^{1.7}$			Double-suction pump, $K = HN_s^{1.7}$		
	Dynamic head, ft.		K, millions	Dynamic head, ft.		K, millions	Dynamic head, ft.		K, millions
	N_s 15,000	N_s 5,000		N_s 3,520	N_s 1,760		N_s 5,000	N_s 2,500	
25	....	....	...	21.8	70	24	21.8	70	42.5
20	5.7	24.1	2.0	35	112	38	35	112	68.5
15	7.8	34	2.8	44	151	49	44	151	89
10	10.1	43	3.6	59	192	64	59	192	116
5	12.1	53	4.4	71	214	75	71	214	135
0	14.4	62	5.2	85	283	94	85	283	170
−5[1]	16.6	72	6.0	93	305	102	93	305	184
−10	18.9	81	6.8	106	340	116	106	340	208
−15	20.5	91	7.5	115	370	126	115	370	225

[1] The negative sign indicates a depth of water in the suction pit over the top of the pump suction.

270. Operation. In starting a centrifugal pump, knowledge of its characteristics may be desirable to avoid overloading of the motor. In general, all centrifugal pumps, except a few low-head pumps with special characteristics at low or no discharge, are best started with the discharge valve closed. Small pumps driven by squirrel-cage motors may be started on reduced voltage if torque is sufficient, and water hammer will not result from surges in the discharge pipe as full-line voltage is developed on the motor. It may be possible to bring the pump slowly up to full load, with the discharge valve closed, by a by-pass provided to return the discharge from the pump to the suction pit. If the pump must be started with the discharge valve open, a wound rotor induction motor developing adequate torque without excessive line current may be used.

Centrifugal pumps must have a submerged suction or must be primed before starting. They can be primed by the use of some form of vacuum pump, such as an ejector or a wet vacuum pump. If a dry vacuum pump is used, a barometric siphon must be provided on its suction to prevent water from getting into it. The pump may be primed also by closing the foot valve and filling the suction pipe and the pump with water. The installation of a typical "self-priming" centrifugal pump is illustrated in Fig. 119. If the pump is noisy, as though filled with gravel, cavitation may be occurring, as explained in Sec. 269. Where there is high suction lift and changes in temperature of the water, "flashing" may occur in the suction line, producing the effects of cavitation or causing loss of prime.

Installations of centrifugal pumps involving automatic operation or remote control are successful.[1] A typical installation, with necessary appurtenances, is shown in Fig. 119. Among the requirements for successful automatic operation are a priming device, a device to limit the number of false starts in the event priming fails, and a cutoff in the event the check valve closes while the pump is operating.

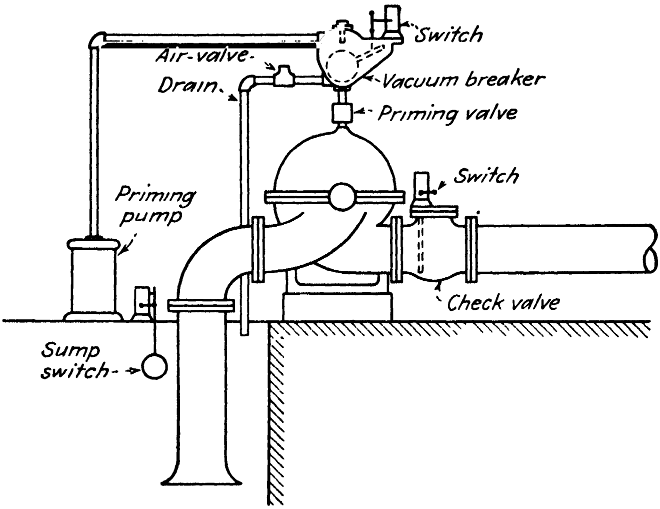

FIG. 119 Self-priming centrifugal pump and automatic control system.

After the pump is started, if no water is delivered the difficulty may be due to (1) the pump not being primed, (2) too low a speed, (3) discharge head or suction head being too high, (4) a plugged impeller, or (5) mechanical difficulties. If the discharge is less than it should be there may be, in addition to the above difficulties, air leakage into the suction side or the suction pipe or the foot valve may be too small. If the motor heats, the speed may be too high, the operating conditions may be off the highest efficiency, or there may be mechanical defects, such as the bearings being too tight.

While running, the pump should be operated under conditions of discharge, head, and speed to give the highest efficiency unless other conditions of operation are more desirable than the greatest economy.

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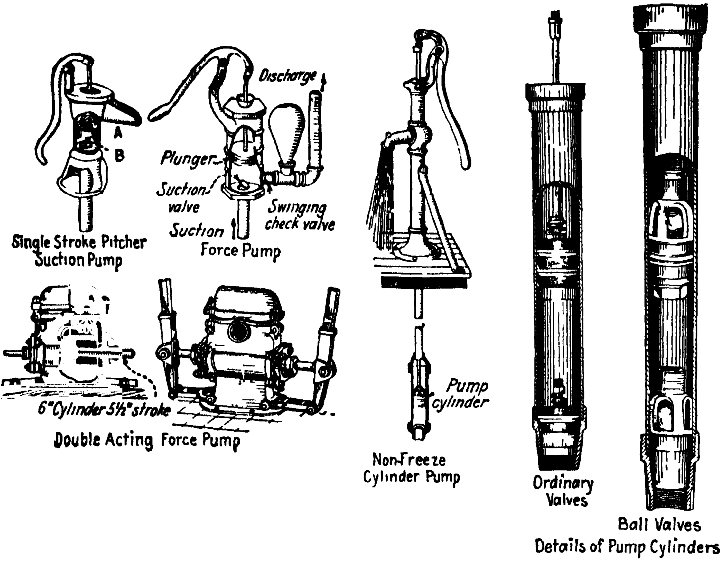
[1] See also HAENTJENS, W. D., "Automatic Operation of Centrifugal Pumps," *Water & Sewage Works*, April, 1946, p. R-78

Owing to wear and other factors the pump characteristics may change materially so that tests of the characteristics should be made periodically. The performance of the pump should be controlled by changing the speed rather than by manipulation of the discharge valve alone. Opening of the valve to decrease the discharge head or closing it to decrease the rate of discharge may increase the load on the motor or decrease the efficiency of the pump and the motor.

CHAPTER XVI

WELL PUMPS

271. Types. Pumps for removing water from wells may be classified as reciprocating, jet, revolving vertical-shaft, and air-lift. Reciprocating pumps may be classified as single-stroke, two-stroke, differential-plunger, and double-acting cylinder pumps. Small reciprocating pumps for private water supplies may be hand-driven or power-driven. Larger reciprocating pumps and all other types of pumps for public water supplies are power-driven. Revolving vertical-shaft pumps may be classified as centrifugal or turbine, mixed-flow, angle-flow, and propeller or screw pumps. The centrifugal or turbine pumps dominate the field in deep wells, the use of reciprocating pumps being restricted to small and to old installations. Jet pumps are used in small and moderate-size capacities. Air lifts are used, under conditions in which they alone are most suitable, in installations of any capacity desired.



Single Stroke Pitcher Suction Pump

Force Pump

Double Acting Force Pump

Non-Freeze Cylinder Pump

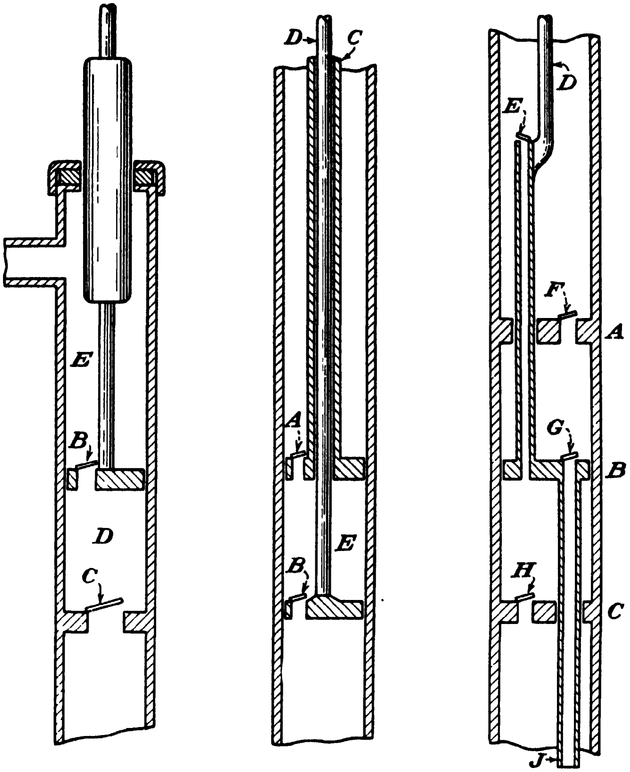
Ordinary Valves

Ball Valves

Details of Pump Cylinders

FIG 120 Types of hand-operated pumps.

272. Single-stroke Pumps. Two hand-operated single-stroke well pumps are illustrated in Fig. 120. The single-acting pump operates as follows: When the handle is pressed down, plunger *A* rises, valve *A* closes, and valve *B* opens. Water above plunger *A* is discharged from the pump, and water is sucked into it below piston *A* through valve *B*. As plunger *A* descends, valve *B* closes, holding water in the pump. In the force pump, water is sucked upward into the pump on the downstroke of the handle and is discharged upward through the discharge pipe on the upstroke. Capacities of hand-driven pumps cannot be expected to exceed about 5 g.p.m.



DIFFERENTIAL PLUNGER PUMP

TWO-STROKE PUMP

DOUBLE-ACTING CYLINDER PUMP

Fig. 121.—Deep-well reciprocating pumps.

273. Two-stroke Pumps. A two-stroke pump is illustrated in Fig. 121. It operates as follows: The rods *C* and *D* move in opposite directions, one moving down as the other moves up. As *D* moves up, valve *B* is closed, and the water in chamber *E* is discharged through valve *A*, one-half of it being discharged from the pump. As rod *C* moves up, the water, which has previously passed through valve *A*, is discharged from the pump, and the chamber *E* is refilled through valve *B*.

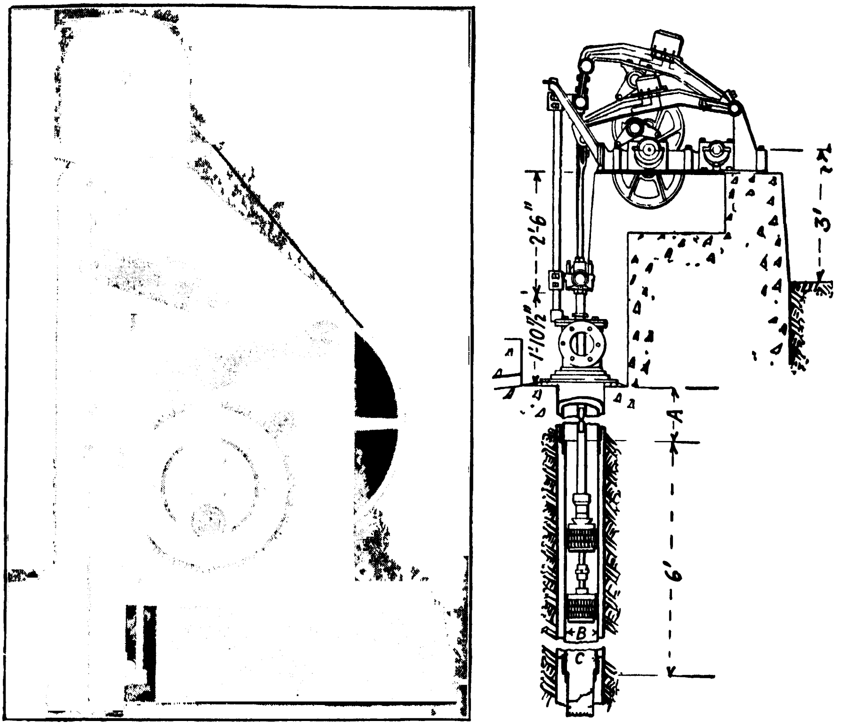
274. Differential-plunger Pumps. A differential plunger pump is illustrated in Fig. 121. It operates as follows: As piston *B* moves up, water is sucked into chamber *D* through valve *C* and water is discharged from chamber *E* through the discharge pipe. The volume of water discharged is equal to the difference of the displacement of the piston *B* and of the differential plunger. On the downstroke, valve *C* closes and the water in chamber *D* passes through valve *B* into chamber *E*. Water is discharged through the discharge pipe, the volume of water discharged being equal to the displacement of the descending differential plunger.

The size of the piston and of the plunger can be proportioned to give equal volumes of discharge, or equal work, on the upstroke and on the downstroke.

275. Double-acting Cylinder Pumps. A section through a double-acting cylinder pump is shown in Fig. 121. *A* and *C* represent fixed partitions across the working barrel. *F* and *H* are check valves in these partitions. *B* is a moving partition or piston attached to the pump rod *D*. *E* and *G* are check valves over the channels through the pump rod. On the downstroke *F* and *H* are closed; *E* and *G* are open. Water is being drawn into the pump through *J* and is being discharged through *E*. On the upstroke *E* and *G* are closed; *F* and *H* are open. Water is being drawn into the pump through *H* and is being discharged at *F*. A disadvantage of the double-acting pump is evident when long rods are used. The thrust of the downstroke tends to bend the rods out of alignment and hence requires the use of guides.

276. Power Heads for Reciprocating Pumps. In reciprocating deep-well pumps the pump, or water, valves are in the well and the power head is at the ground surface, connected to the valves by long slender rods that rise and fall in the well. Two types of power heads are shown in Fig. 122. The motion of the water valve actuated by the power head shown in Fig. 122*A* is known as harmonic motion. The velocities and the time of upward motion have the same magnitude and variations as those during the downstroke. The cam shown in Fig. 122*B* makes the upward motion slower. The upward motion therefore lasts longer than the downward; when it is used to drive a two-stroke pump, one valve is always rising. This action reduces the magnitude of velocity changes in water passing through the well. The result is greater efficiency, smoothness of action, and longer life to the pump.

277. Jet Pumps. A jet pump is illustrated in Fig. 123. It consists of a centrifugal pump and motor at the ground surface and a jet down in the well below the water level. The centrifugal pump has two discharge pipes. One discharge pipe leads water downward to the jet through which it is discharged at a high velocity which serves to lift water into the suction of the centrifugal pump. Jet pumps are available in capacities from 70 g.p.m. at 30-ft. lift to about 20 g.p.m. at 150-ft. lift. They are widely used in low-capacity installations, but they are too small for general use in public water supplies.



(*A*) (*B*)

Fig. 122.—Deep-well pumps (*A*) two-stroke pump, (*B*) Pomona deep-well pump. (*Courtesy of American Well Works.*)

278. Revolving Vertical-shaft Pumps. There are four types of revolving vertical-shaft pumps illustrated in Fig. 124. A two-stage turbine pump is shown in greater detail in Fig. 125, and a more complete installation is shown in Fig. 126. This type of pump is used almost exclusively in waterworks practice. Six-inch wells are about the smallest in which such pumps can be used. They are available in all sizes above about 5½-in. impellers up to pumps with capacities above 1,000 g.p.m. Almost any reasonable head, or lift, required can be obtained by increas-

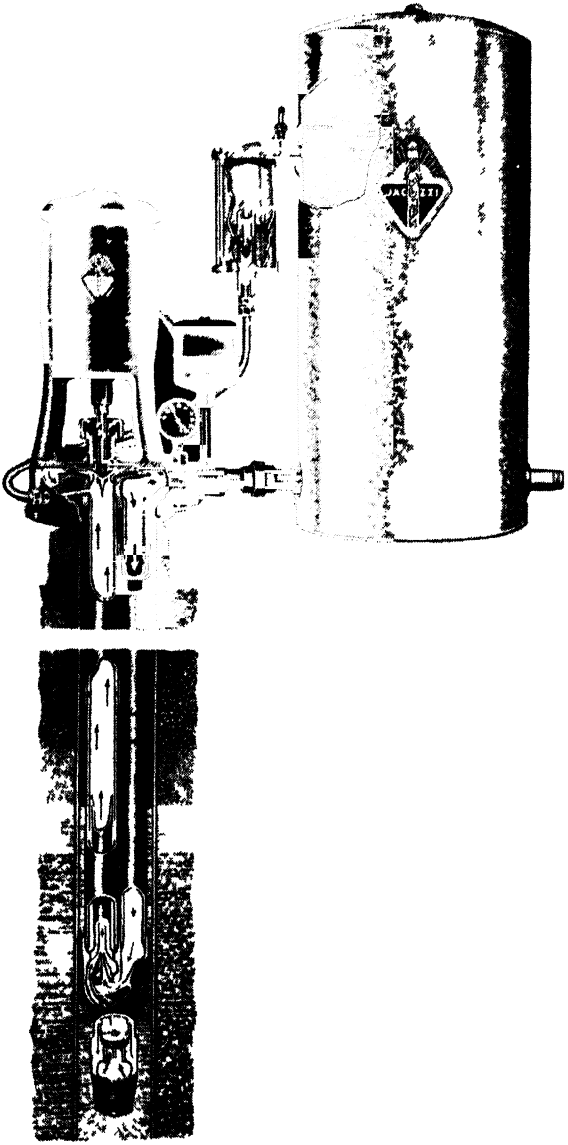
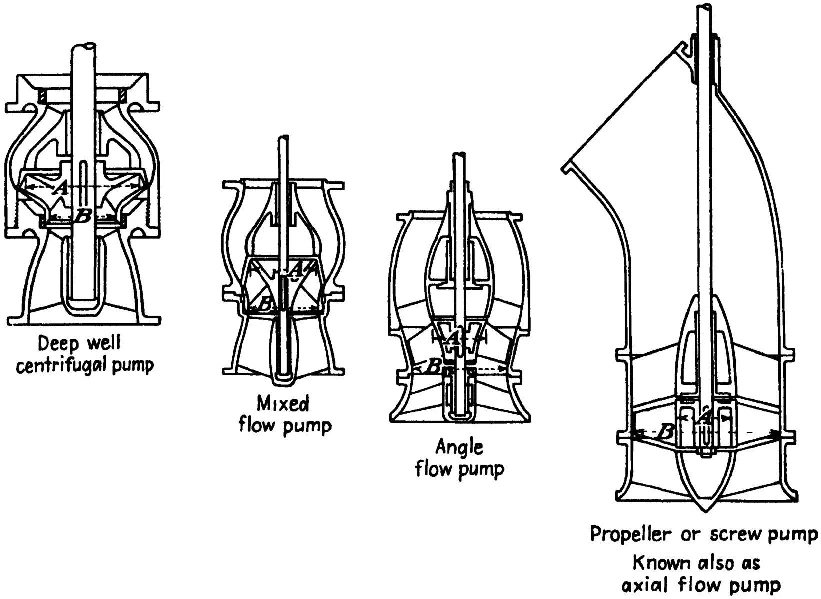


Fig. 123 —Typical cross section of two-pipe jet pump with pump set directly over the well (*Courtesy of Jacuzzi Bros, Inc*)

ing the number of stages.[1] Efficiencies up to 90 per cent, and better, are reported, but only 75 to 80 per cent, or lower, should be anticipated in capacities less than about 200 g.p.m.

Thrust bearings at the bottom of the shaft carry the full weight of pump parts, shaft, and the unbalanced weight of the rising water. The lubrication of thrust bearings and of shaft bearings presents a difficult problem in the design and operation of the pump. Present practice tends toward oil rather than water lubrication.[2] To avoid the difficulties with the long shaft required where the motor is at the top of the well and the pump near the bottom, a submersible unit has been used in which the motor is placed below the pump in the well and is connected directly to the centrifugal pump. The entire unit is submerged in the well, the suction strainer lying between the motor and the pump.[3] The motor, which is of the squirrel-cage type, operates in an enclosed housing, in a bath of oil, protected from the water. The economy of the unit, when



Deep well centrifugal pump

Mixed flow pump

Angle flow pump

Propeller or screw pump Known also as axial flow pump

FIG. 124.—Four types of deep-well centrifugal pumps.

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[1] See also CONANT, D. J., "Turbine Well Pumps—Their History and Development," *Jour. A.W.W.A.*, October, 1932, p. 1499.

[2] See also WARD, C. N., "Deep Well Pumping," *Jour. A.W.W.A.*, Vol. 28, p. 361, 1936; and "Lubrication of Water Pumping Equipment," *Public Works*, January, 1936, p. 30.

[3] "Submerged Motor Used in Deep Well Pumping Unit," *Water Works Eng.*, Nov. 24, 1937, p. 1684.

compared with the more common arrangement of pump and motor, becomes apparent for depths of well greater than about 100 ft.

Helical or axial-flow pumps are available in lifts up to 600 ft. with capacities up to 50 g.p.m.

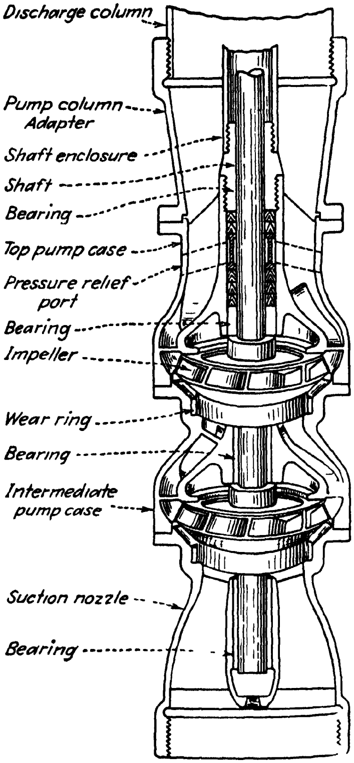

FIG. 125.—Two-stage deep-well centrifugal pump. (*From Water & Sewage Works, April,* 1946, *p. R-88.*)

FIG. 126. Deep-well centrifugal pump (*Courtesy of Peerless Pump Division, Food Machinery Corporation.*)

High-lift revolving vertical-shaft pumps are suitable for pumping directly from the well into the distribution system or into elevated tanks where their economy for this service exceeds that of pumps lifting water into a surface reservoir from which it is pumped to the distribution system by another type of pump.

279. Selection of Turbine Pumps.[1] Materials of construction, quality of water, relation between drawdown and discharge from the well, specific speed, and other conditions must be considered in the selection of a deep-well turbine pump. Where water is corrosive, corrosion-resistant materials should be selected.[1] For example, a corrosion-resistant installation would be represented by a stainless-steel shaft, bronze bowl or shell and impellers, stainless-steel line shaft from bowls to static water level, with a 5-ft. section of bronze oil tubes, with coal-tar enamel on the inside and on the outside of the eduction or column pipe. No installation can long resist erosion by sand-bearing water or by cavitation. In general, the higher the speed, the faster the corrosion.

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[1] See also FABRIN, A. O., "Selecting Deep Well Centrifugal Pumps," *Water & Sewage Works*, April, 1946, p. R-87.

The pump characteristic must be suited to the drawdown in the well for the rate of discharge and lift desired. The characteristic and the drawdown may be coordinated by constructing productivity curves and characteristic curves of the well and pumps under consideration, as illustrated in Fig. 127.[2] The curves show that both pumps *D* and *E* are best suited to draw 700 g.p.m. from the well with a total lift of 145 ft. but that pump *D* is better suited for all conditions since it more nearly follows the range of most economical production and has a higher capacity under low-water conditions during the summer when more water is needed.

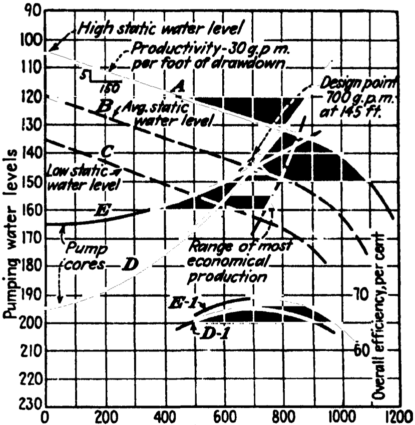

FIG. 127.—Typical pump characteristics used in selection of deep-well pump. (*From L. J. Alexander, Jour. A.W.W.A., October, 1946, p. 1205.*)

Specific speed (N_s) is an important consideration in the selection of vertical rotating shaft pumps. Fabrin[3] has shown that where N_s is less than about 4,100 the deep-well centrifugal pump should be used; between 4,100 and 5,800 a mixed-flow pump, between 5,800 and 7,500 an angle-flow pump, and above 7,500 the propeller or screw pump should be used. The most favorable designs will be found at a specific speed of 2,500, with a favorable range between 1,500 and 4,100. Specific speed is related to the ratio of wheel diameter *D* and to inlet diameter *B*. This relationship has been formulated empirically as

$$\frac{D}{B} N_s^{0.53} = 10{,}600 \qquad (1)$$

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[1] See also ALEXANDER, L. J., "Purchase and Maintenance of Deep Well Pumps," *Jour. A.W.W.A.*, October, 1946, p. 1197.

[2] *Ibid.*

[3] FABRIN, A. O., *loc. cit.*

when N_s is less than 4,100 and D/B is more than 130

$$\frac{D}{B}N_s = 530{,}000 \tag{2}$$

when N_s is more than 4,100 and D/B is less than 130

It is to be noted that when N_s is high, D/B is low and the condition is unfavorable to high suction lift.

An empirical formula, based on the recommendations of Fabrin, showing the relation between some controlling conditions in pump selection can be expressed as

$$D = \frac{236Q^{0.154}H^{0.256}}{N^{0.678}} \tag{3}$$

where D = outside diameter of pump casing, in.
H = total head, in feet of water, for one stage or wheel
N = speed of the wheel, r.p.m.
N_s = specific speed $= \dfrac{N\sqrt{Q}}{H^{3/4}}$
Q = rate of discharge, g.p.m.

The formula is limited to pumps of the highest efficiency, with specific speeds between 4,100 and 1,500.

If, in the selection of pump diameter, the computations show a diameter approximately midway between two available sizes, the larger size will probably give better life and efficiency than the smaller size. The following is an example of the computations required in the selection of pump size, speed, and stages.

Example: Let it be desired to select a turbine deep-well pump; when the well is to be constructed to fit the pump, the rate of discharge is to be 750 g.p.m. and the total lift is to be 200 ft.

Solution: Assuming a convenient specific speed of 2,500, determine the speed of the pump.

$$N = \frac{N_sH^{3/4}}{\sqrt{Q}} = \frac{2{,}500 \times 51.6}{27.4} = 4{,}710 \text{ r.p.m.}$$

This speed is too fast. To reduce it, assume four stages, reducing the head to 50 ft. per stage. Then

$$N = \frac{2{,}500 \times 18.8}{27.4} = 1{,}715 \text{ r.p.m.}$$

The speed is satisfactory.
From formula (3)

$$D = \frac{236 \times 750^{0.154} \times 50^{0.256}}{1{,}715^{0.678}} = 11\tfrac{1}{2} \text{ in.}$$

Hence, a four-stage, 11½-in. casing turbine pump is probably the best for the conditions, provided its characteristics fit the drawdown in the well.

If a 10½-in. casing is the nearest size available, or the diameter of the well restricts the pump to this size, then the problem will be to determine the speed at which the pump must run and its specific speed, to meet the conditions for a turbine pump.

To determine the new speed, transposing formula (3)

$$N = \left(\frac{236Q^{0.154}H^{0.256}}{D}\right)^{1/0.678} = \left(\frac{236 \times 750^{0.154} \times 50^{0.256}}{10.5}\right)^{1.476} = 2{,}000$$

and

$$N_s = \frac{2{,}000 \times 750^{0.5}}{50^{3/4}} = 2{,}910$$

Since the speed of 2,000 r.p.m. is somewhat high, it must be reduced. This can be done by increasing the number of stages. At six stages the speed becomes 1,660 r.p.m. and the specific speed becomes 3,260. Both of these are within the desirable range for a turbine pump. Again, the characteristic of the pump selected must be coordinated with drawdown conditions in the well.

280. Air-lift Pumps. An arrangement of an air-lift pump is shown in Fig. 128. It operates as follows: Compressed air is released through a "foot piece," or air diffuser, at the bottom of the eductor pipe and, rising in small bubbles, a mixture of air and water is created that has a lower specific gravity than that of water alone. The weight of the column of air and water from A to B in the figure equals the weight of the column of water from A to C. The rising air bubbles, if sufficiently small, exert no appreciable lifting force on the water. If the air bubbles are large enough to move upward more rapidly than the rising water, efficiency is lost. Unfortunately there is a tendency for small air bubbles to coalesce and form larger bubbles that rise more rapidly than the water and result in intermittent gushing discharge. It is to be emphasized that water is not pushed from the well by rising pistons of air.

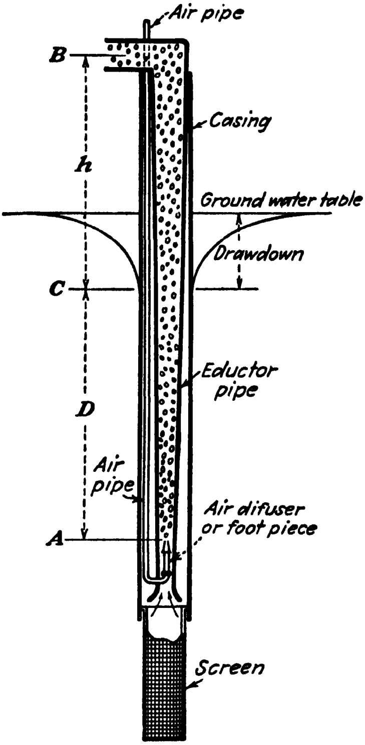

Fig. 128.—Air-lift pump.

Air lifts have many unique features not possessed by other types of well pumps. They are the most simple and foolproof type of pump and, in operation, give the least trouble because there are no remote or submerged moving parts. Air lifts can be operated successfully in holes of any practicable size, and they can be used in crooked holes not suited to any other type of pump. If it were not for the air lift, crooked holes would be valueless. An air-lift pump can draw more water from a well, with sufficient capacity to deliver it, than any other type of pump that can be installed in the well. A number of wells in a group can be operated from a central control station where the air compressor is located.

The principal disadvantages to the universal adoption of air lifts as deep-well pumps are the necessity for making the well deeper than is required for other types of well pumps in order to secure adequate submergence of the air outlet; the intermittent nature of the flow from the well; and the relatively low efficiencies developed. As the efficiency is greatly affected by both the lift and the submergence, the dropping of the water level in the well without corresponding changes in the well equipment and construction is a common cause of gross inefficiency in the operation of air-lift pumps. Little is usually known of the efficiency of the average air-lift installation in small waterworks, but tests have revealed efficiencies in the neighborhood of 45 per cent for depths of 50 ft., down to 20 per cent for depths of 600 ft. The changes in the efficiencies of an air lift resulting from the use of different submergence ratios are shown in Table 51.

TABLE 51.—EFFECT OF SUBMERGENCE ON EFFICIENCIES OF AIR LIFT AT HATTIESBURG, MISS.

Ratio D/h	8.70	5.46	3.86	2.91	2.25
Submergence ratio $= D/(D + h)$	0.896	0.845	0.795	0.715	0.693
Percentage efficiency	26.5	31.0	35.0	36.6	37.7

Ratio of D/h	1.86	1.45	1.19	0.96
Submergence ratio $= D/(D + h)$	0.650	0.592	0.544	0.490
Percentage efficiency	36.8	34.5	31.0	26.5

281. Design of an Air Lift.[1] The factors entering into the design of an air lift include (1) Q_w, the rate at which water is to be raised in cubic feet per second; (2) h, the effective lift from the "free" surface of water outside the well casing in feet when pumping (see Fig. 128); (3) D, the depth of submergence in feet. This is the depth at which air is liberated in the well below the "free" surface of water outside the eductor pipe. (4) $l = D + h$, expressed in feet, represents the effective length of the eductor pipe; (5) Q_a, the rate at which free air is forced into the well, expressed in cubic feet per second; (6) H, the total head pumped against. This is equal to the velocity head, plus static lift, plus all friction losses expressed in feet. (7) r, the ratio of compression, which equals $(D + 34)/34$. The design of an air lift usually requires the determination of

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[1] WARD and KESSLER, "Experimental Study of Air Lift Pumps and Application of Results to Design," *Univ. Wis. Bull.*, 1924. IVENS, E. M., "Pumping by Compressed Air," 2d ed., John Wiley & Sons, Inc., New York, 1920. PICKERT, F., "The Theory of the Air Lift," *Engineering*, July 1, 1932, p. 19.

1. The depth of the well.
2. The capacity of the air compressor.
3. The size of the air pipe.
4. The starting and operating pressures.
5. The size of the eductor pipe.

These are all based upon the rate at which water is to be pumped and the height to which it is to be lifted.

1. *The Depth of the Well.* In order that water may be lifted from a well, it is necessary that air be released at some depth below the surface of the water. It is evident in Fig. 128 that the weight of the column of mixed air and water within the eductor pipe must equal the weight of the column of water, of the same cross-sectional area, outside the eductor pipe. It is evident, therefore, that as h is increased D must be increased and that there is some relation between them for impending delivery. The magnitude of D can be greater than is necessary to cause delivery, but the efficiency of the pump will be affected because of the increased friction in the longer eductor pipe. Because of the velocity imparted to the water by the rising bubbles, D may be slightly less than the amount necessary to cause perfect balance in order to lift water. The value of D is known as the *submergence*, and the expression $D/(D + h)$ is the *submergence ratio*, or the percentage submergence, when multiplied by 100.

2. *The volume of free air* required can be expressed, approximately, as

$$Q_a = \frac{Q_w(h + h_1)}{75\bar{E} \log r} \tag{4}$$

where h_1 = velocity head at discharge, usually taken as 6 ft. for deep wells, down to 1 ft. for shallow wells.

E = efficiency, usually about 45 to 50 per cent with submergence ratio of 50 to 65 per cent and submergence between 350 and 500 ft

In any case some excess air capacity should be provided, because if the free water surface in the well should fall more than anticipated after prolonged pumping, more air will be required to maintain the discharge.

3. *The Size of the Air Pipe.* The size of the air pipe depends on the rate at which air is to be transmitted and the velocity of flow permitted in the pipe. No acceptable formula for the economical velocity of flow in an air pipe is available. The velocity assumed is usually between 1,000 and 2,000 f.p.m. The size of the air pipe can be computed from the expression $d = 13.6(q_1 \div V)^{1/2}$ and the loss of pressure computed from the following formula:[1]

$$P_1^2 - P_2^2 = \frac{0.0006Q_a^2l}{d^5} \tag{5}$$

where P_1 = absolute initial pressure, p.s.i.
P_2 = absolute terminal pressure, p.s.i.
q_1 = volume of air in cubic feet per minute passing through pipe
Q_a = free-air equivalent in cubic feet per minute of volume passing through pipe
l = length of pipe, ft.
d = diameter of pipe, in.
V = velocity of flow, f.p.m.

If the size of the pipe and the loss of head are reasonable, no furthur computations are necessary. If not, the size of the pipe must be recomputed.

4. *Starting and Operating Pressures.* The starting pressure is equal to the depth of water over the submerged end of the air pipe or foot piece. After the pump has been operating a sufficient length of time for the water in the well to reach constant conditions, the operating pressure will become constant and will be equal to the friction loss in the air pipe plus the depth of water outside the well casing and above the foot piece. The latter is the same as the difference between the depth of water in the well when not pumping and the drawdown for the particular rate of discharge.

5. *The Size of the Eductor Pipe.* It is desirable to have a high velocity in the eductor pipe in order to prevent the formation of slugs of air and water. The velocity of the mixture of air and water should increase in ascending the pipe in order to avoid, as much as possible, "slip" of the bubbles. The maximum efficiency has been found to occur near that velocity at the lower end of the eductor pipe at which the mixture first becomes complete. This velocity varies with the diameter of the eductor pipe only, so far as is now known. Experiments have shown the following velocities to be suitable:

Size of air pipe, in.	2¾	3	4	5
Velocity at entrance to eductor pipe, f.p.s.	5.6	7.3	11.8	12.7

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[1] JOHNSON, J. E., *Am. Machinist*, July 27, 1899.

The velocity at discharge is assumed to be about 20 to 25 f.p.s. The volume of the mixture must be computed as the volume of the water plus the volume of the air corrected for the proper pressure.

282. Air-lift Boosters. The inefficiency of air lifts renders them unsuitable for lifting water after it has been delivered from the well, particularly for moving the water in any direction but vertical. Because of the high velocity of the flowing water at the discharge, there is considerable kinetic energy that can be utilized for further lifting the water, thus recovering some energy that otherwise would be dissipated.

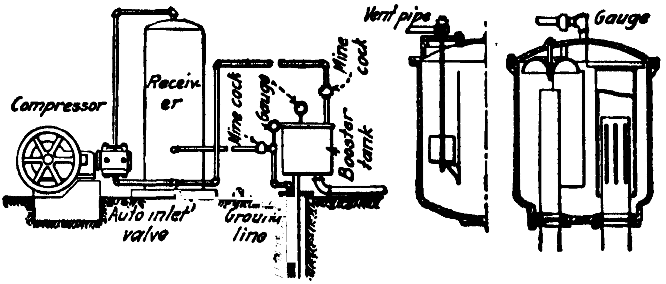

FIG. 129.--Sullivan air-lift booster.

The device used for this purpose is known as a booster. It is illustrated in Fig. 129. The air and water escaping from the eductor pipe are separated by the umbrella, the water falling to the bottom of the container, the air rising to the top. The entrapped air is compressed partially by the water discharging from the well. This forms a back pressure on the well and a pressure on the entrained water which forces it out through the slotted discharge pipe. Air escapes continuously through the air vent at the top. The air vent is throttled to maintain the desired pressure in the booster.

If the air from the booster is returned to the compressor, an advantage is gained in the use of cooler air and a slight increase in pressure of the air at the intake.

CHAPTER XVII

MISCELLANEOUS ENGINES AND PUMPS

283. Internal-combustion Engines. An internal-combustion engine develops power by the explosion of fuel within the cylinder of the engine. Internal-combustion engines are particularly suited as stand-by equipment in pumping stations depending primarily on electric power generated elsewhere than in the pumping station. In the event of interruption of electric service, the internal-combustion engine can be started relatively quickly, with little or no interruption of service. The best known types of such engines are the gas or gasoline engine, operating on the Otto cycle, and the diesel engine.

284. Gas or Gasoline Engines. An outstanding advantage of the Otto-cycle engine is its quick-starting characteristics. The speeds of such engines are suited for direct drive of electric generators or centrifugal pumps. The first cost of such engines and their maintenance are usually lower than the corresponding costs of diesel engines of equal power. However, the fuel economy of Otto-cycle engines, compared with other available power sources, is usually such that gas or gasoline engines are not often used as the main power-generating units in a waterworks pumping station.

In a comparison of the features of two-cycle and four-cycle engines it is found that the latter can operate at a higher speed; the crankcase need not be enclosed; better lubrication is secured because of the lower temperatures; the economy and efficiency are higher; there is less loss of energy in exhausting old and inhaling new gases; and the power per unit volume of piston displacement is only about 52 to 58 per cent of that of a two-cycle motor, because there is only one power stroke in four rather than in two. In the two-cycle motor the exhaust gases are less completely expelled; there are fewer valves; and there is less weight per horsepower, which requires less space and lighter foundations.

High-speed gasoline engines are low in first cost but are relatively high in fuel cost compared with diesel engines. Large gas engines will develop 1 hp.-hr. on 0.7 lb. of fuel at full load. Units are available up to 585 hp., and, as they operate at speeds of 720 to 1,500 r.p.m., they are suitable for direct connection to centrifugal pumps or electrical generators.

285. Diesel Engines.[1] Diesel engines have high thermal efficiency; they can use almost any type of liquid or gaseous fuel; they are expensive in first cost; they are not easy to operate; they are difficult to start, requiring an outside source of power to compress the air to the proper firing temperature in the cylinder; and their speed is slow, usually requiring a speed-increasing mechanism when driving either an electric generator or a centrifugal pump. The use of speed-increasing gears on reciprocating engines has not been extensive. Common forms of power storage used for the purpose of starting diesel engines include electric storage batteries and tanks of compressed air. Other difficulties with the operation of diesel engines are caused by the relatively large number of moving parts and the experienced care required in their lubrication and maintenance.

Diesel engines have good partial-load characteristics and good speed regulation. They will require about 0.48 lb. of crude oil per brake horsepower-hour,[2] whereas a semi-diesel engine will consume about 10 per cent more. The fuel efficiency is quite constant over a considerable variation of speed and head, and the economy of small units is almost equal to that of the largest sizes, making possible the use of a number of small units instead of concentrating into units of the largest possible size, as is proper for steam.

The diesel engine is approximately 35 per cent more economical at full load than the Otto engine, on the basis of pounds of fuel per horsepower developed, and 50 per cent more economical at light loads. In general terms the economy is in the ratio of 2:3. Fuel at present is sold by volume instead of by weight; therefore the diesel effects a further saving compared with gasoline-driven engines in the ratio of approximately 7:8. The price of gasoline to the average user is about 3 or 2:1 compared with fuel oil. Therefore the over-all operating cost of fuel for a diesel engine is 20 to 25 per cent of that for an equivalent Otto gasoline unit. Lubricating costs are practically the same for the two types of engines. The fuel efficiency of a diesel engine is about 32 per cent as compared with the best steam-driven thermal efficiency of about 28 per cent.[3] It is, therefore, plainly evident that the total annual hours of service, representing a definite saving, becomes a deciding factor to the purchaser in his selection of power.[1] Couch[2] has made a valuable comparison of steam, electric, and diesel power for waterworks pumping stations.

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[1] See also RUSSELL, W. S, "Diesel Engines in the Water Works Field," *Jour. A.W W.A* , January, 1936, p 119; and BOEHM, A. M., "Internal Combustion Engines in Water Works and Sewage Plants," *Jour. New Engl. Water Works Assoc.*, Vol 60, p 27, 1946.

[2] See also McDONALD, N G , "Modern Trends in Pumping Station Equipment," *Jour A W W A* , August, 1942, pp 1241 and 1253

[3] See also EARL, E , *Jour New Engl Water Works Assoc* , Vol 47, p 61, 1933.

Diesel-engine units are made in sizes larger than Otto-cycle engines. The 6,850 hp. units in the municipal power station at Vernon, Calif., are among the largest units made.[3] The advantages of the diesel engine so outweigh its disadvantages that it has taken a permanent place in the waterworks field.[1]

286. Diesel Auxiliaries. The principal auxiliaries required in connection with a Diesel engine have to do with the handling of the fuel oil.[4] They include a hand-operated and a power-operated fuel-oil pump, cooling-water equipment, oil-storage equipment, equipment for warming the fuel-oil, oil meters, and oil purifiers.

287. Fuel Oil. Although diesel engines will operate with many different kinds of liquid and gaseous fuels, the efficient operation of any particular engine requires discriminating selection of the fuel. It is desirable that the type of engine installed shall be suited to the type of oil available in the locality. As a general guide in the purchase of fuel oil the following desirable characteristics may be considered: (1) minimum specific gravity, 24°Bé; (2) viscosity (Saybolt Universal at 100°F.), minimum 40 sec., maximum 100 to 150 sec.; (3) sulphur, maximum 1.5 per cent; (4) Conradson carbon (carbon residue), 1.0 per cent; (5) flash point, minimum 150°F.; (6) water and sediment, by volume, 0.5 per cent; (7) pour point, less than the minimum temperature at which the oil is to be used. Gravity, flash point, and sulphur should, in general, be given the least consideration provided the flash point is above the legal limit.

288. Water Power and Wind Power. The development of hydraulic power for waterworks purposes is a neglected field, because conditions are not usually favorable for its development and utilization, other forms of power being less expensive.

For small installations for private estates and remote institutions the hydraulic ram may be used for raising water from a falling stream, or the windmill may be used for well pumping or surface-water pumping. A form of hot-air engine that has been used occasionally for small installations is now almost obsolete. Except for the hydraulic ram, which is manufactured with capacities up to 700 g.p.m., such devices are seldom used to raise more than a few gallons of water per minute and are not used for lifting public water supplies. Where water power is available, water turbines may be successfully connected directly to centrifugal pumps, as at Calgary.[1]

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[1] See also JAHNKE, C. B., *Assoc. State Eng. Socs. Bull.*, April, 1934, p. 7.

[2] See *Jour. New Engl. Water Works Assoc.*, Vol. 44, p. 458, 1930.

[3] See *Civil Eng.*, September, 1935, p. 533.

[4] See also HALL, R. D., *Jour. A.W.W.A.*, Vol. 23, p. 337, 1931.

289. Power Pumps. A pump that is connected to the driving engine by means of gearing, belting, or shafting is called a power pump, provided that it is not integrally connected to the motive power as in a steam pump.

The triplex pump, illustrated in Fig. 130, consists of three water cylinders operated in parallel by means of some type of power drive. It is a frequently used type of power pump and gives satisfactory service.

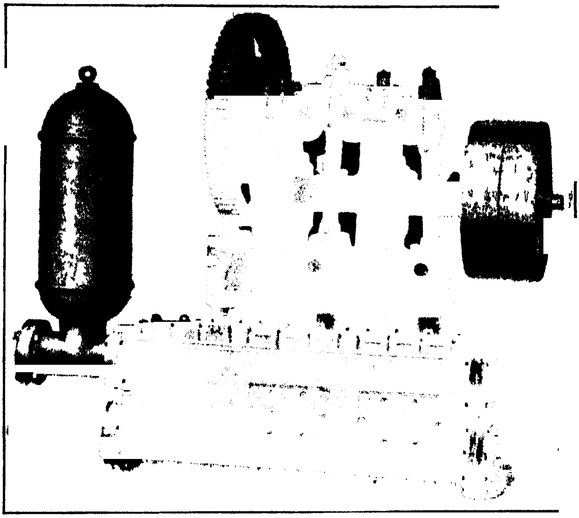

Fig. 130.—Triplex power pump. (*Deming*)

Such pumps are available in capacities up to about 6,000,000 gal. per day and under about 200 hp. With first-class maintenance they will develop efficiencies up to about 85 per cent. In the installation of a triplex pump some form of pressure relief should be connected to the discharge pipe so that in the event of a stoppage in the discharge line, bursting pressures will not be developed. The rigid connection between the pump and the driving engine allows of no pressure relief in the pump.

Types of power pumps, which may also be used as the water end of steam pumps, are illustrated in Fig. 131. Two types of single-acting piston pumps are available. In the type illustrated in Fig. 131*A* the water does not pass through the piston or cylinder, whereas in the type shown in Fig. 131*B* the water flows through a port in the piston. The latter is used mainly for well pumps.

In each of the single-acting pumps the motion of the water is alternately started and stopped, resulting in vibration, shock, and loss of energy. This condition is partly overcome by the U-pump shown in Fig. 131*C* in which the flow of water is always in the same direction.

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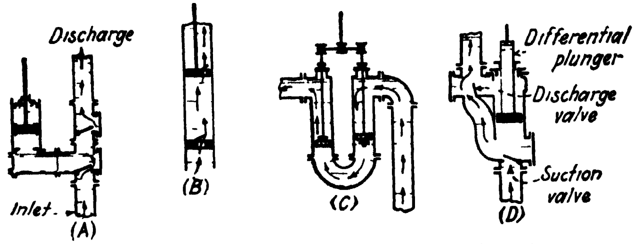
[1] See also McDONALD, *loc cit.*

When the discharge pipe and pump chamber of the differential-plunger pump shown in Fig. 131*D* are filled with water, the discharge valve closes as the plunger rises, and water is drawn into the pump through the suction valve. At the same time the water between the plunger and the walls of the cylinder is expelled through the discharge pipe. There is, therefore, both suction and discharge on the upstroke of the plunger. On the downstroke the suction valve is closed, and water is expelled from the pump chamber through the discharge valve. A portion of this water leaves through the discharge pipe, and another portion passes into the space between the plunger and the cylinder walls.



Single Acting Piston Pumps U Pump Differential Plunger Pump

FIG 131 – Types of pump.

290. Jet Pumps. The jet pump shown in Fig. 132 operates because the velocity of water passing through the jet is retarded by the expansion of the stream beyond the jet orifice. The increase in the size of the stream creates a partial vacuum which raises water through the suction pipe. The velocity energy of the incoming high-pressure water is converted into pressure energy in the discharge pipe. A jet pump is simple and reliable and requires no lubrication and little or no attention in operation. Such pumps are suitable only where the mechanical equipment must be of the simplest kind and where high-pressure water is available at a low cost.

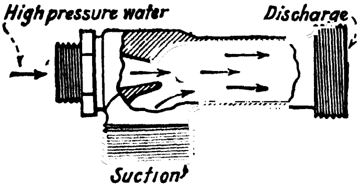


Diagram of Interior

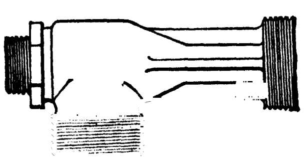


FIG. 132.—Hydraulic ejector.

The discharge lift of the pump is limited to about 12 to 18 ft., and the discharge line should be arranged to prevent the back flow of air. This can be done by submerging the end of the discharge pipe. The operation of the pump requires about 4 to 5 p.s.i. pressure on the high-pressure line for each foot of lift on the discharge line and about 1 gal. of high-pressure water for each gallon of water raised through the suction pipe for lifts up to about 8 ft. In order to obtain the best efficiency the pumps should be so set that all of the lift is in the suction pipe.

Jet pumps are used mainly as auxiliaries in pumping stations, particularly where steam is available. Their discharge pressures and efficiencies are low.

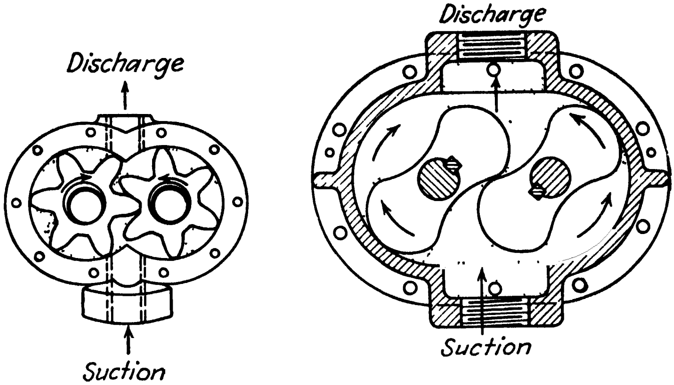


FIG. 133—Rotary pumps.

291. Rotary Pumps.[1] A rotary pump is a displacement pump, since the revolving blade fits close to the casing, thus pushing the water from the pump by displacement. Sections through rotary pumps are shown in Fig. 133. The blades revolve in a downward direction at the center, water being carried upward around the sides of the casing. Water is thus pushed through the discharge pipe, and a partial vacuum is created on the suction side. A relatively high vacuum is created because of the tight-fitting parts.

Rotary pumps are used in small waterworks and to some slight extent in larger units on public water supplies. They are not suitable for handling liquids containing suspended matter because of the close fit of the rotors in the casing. The pumps are self-priming. They give operating efficiencies between 50 and 85 per cent, with a flat efficiency curve under partial loads.[1]

A rotary pump is necessarily power driven and, because of its positive action, provision for pressure relief must be put on the discharge pipe.

292. Miscellaneous Pumps. The many devices used for pumping water testify to the ingenuity of man in their invention. Those devices which are used to some extent in pumping public water supplies are described in other sections of this book. Air-displacement pumps, hydraulically driven reciprocating pumps, Humphrey pumps, diaphragm pumps, and steam vacuum pumps are only a few of the types of machines used for moving water. They are useful primarily in other fields than that of public water supplies. Screw pumps are infrequently used in the pumping of public water supplies because of the low lift to which they are limited. One type of screw pump is similar to a ship's propeller. Another consists of a helix revolving in a closed cylinder open only at the top and bottom.

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[1] BLAKEWELL, W E, "Rotary Pumps," *Power*, November, 1934, p 602

CHAPTER XVIII

MATERIALS FOR AND THE DESIGN OF PIPES

293. Materials. Materials used most commonly for distribution pipes include iron, steel, and cement. Lead, copper, zinc, tin, and alloys such as brass, bronze, and stainless steel are used, in addition to ferrous metals, in pumping machinery, small pipes, valves, and other appurte-

TABLE 52.—SOME STANDARD SPECIFICATIONS FOR WATERWORKS EQUIPMENT

Number	Subject	Published in *Jour. A.W.W.A.*
ASA A21.1 1939	Strength and thickness of cast-iron pipe	Dec., 1939
ASA A21.2 1939	Pit-cast cast-iron pipe	Dec., 1939
ASA A21.4 1939	Cement-mortar lining for cast-iron pipe	Dec., 1939
7C.1 1908	Cast-iron special castings	
7D.1 1938	Laying cast-iron pipe	Feb., 1938
7A.1 1940	Riveted-steel pipe	Jan., 1940
7A.2 1940T[1]	Lock-bar pipe	Jan., 1940
7A.3 1940	Steel water pipe 30 in. and over	Jan., 1940
7A.4 1941TR[1]	Steel water pipe up to 30 in	April, 1943
7A.5 1940	Coal-tar enamel coatings, steel pipe 30 in. and over	
7A.6 1940	Coal-tar enamel coatings, steel pipe up to 30 in.	
7A.7 1941	Cement-mortar coating, pipe 30 in. and over	
7A.8 1946T[1]	Field welding of steel pipe joints	March, 1946
7F.1 1939	Gate valves	March, 1939
7F.2 1941T[1]	Sluice gates	Oct., 1941
7F.3 1940	Fire hydrants	Aug., 1940
7F.3.1 1937	Uniform marking of fire hydrants	April, 1937
7B 1943T[1]	Reinforced-concrete pressure pipe	April, 1943
7H.1 1943	Elevated steel reservoirs	
7H.2 1943	Repainting elevated steel reservoirs	July, 1943
7M.1 1946	Cold-water meters, displacement type	Dec., 1941
7M.2 1946T[1]	Cold-water meters, current type	April, 1946
7M.3 1946T[1]	Cold-water meters, compound type	April, 1946
7M.4 1947T[1]	Cold-water meters, fire service type	Feb., 1947

1931 Federal Specifications for Cast-iron Pipe WW-P-421, not published in *Jour. A.W.W.A.*

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[1] Where only a year appears in this column the specifications have been adopted as standard. The letter T indicates tentative; TR indicates a tentative revision.

nances. Sulphur, sand, rubber, lead, lead substitutes, and cement are used in pipe joints. Other substances such as nickel, aluminum, chromium, clay, and asbestos are used occasionally in alloys or mixtures principally for protection of other metals against corrosion. The Second World War brought out many substitutes for materials formerly used, such as the plastics, some of which have remained to become competitive. In general, standard specifications that have not been changed as a result of war experience are being adhered to. Some standard specifications for waterworks materials are listed in Table 52.

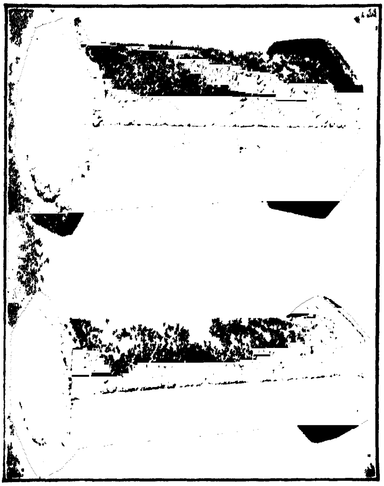

Fig. 134 Cast-iron pipe laid in Versailles, France, in 1664, under Louis XIV, excavated in 1910 (*Courtesy of Cast-iron Pipe Research Association.*)

294. Cast-iron Pipe. Cast iron is the material most commonly used for distribution pipes because of centuries of satisfactory experience with it. Its durability under conditions ordinarily encountered is exceeded by no other material. The condition of an ancient cast-iron pipe used for the fountains at Versailles, France, is shown in Fig. 134.

295. Manufacture of Cast-iron Pipe., Cast-iron pipe is manufactured by two methods: (1) in sand molds, and (2) by the centrifugal process. In sand casting, molten metal is poured into sand-lined molds that are set in either a vertical or horizontal position. Horizontally cast pipe is often referred to as "McWane pipe." Pipe cast in the vertical position is usually referred to as "pit cast." Pipes 18 in. or more in diameter are cast with the bell end down in order to avoid weakness from the accumulation of slag in the bell portion. Smaller pipe may be cast with either end down. Pit-cast pipe is manufactured in accordance with American Water Works Standard Specifications for Cast Iron Water Pipe and Fittings, adopted May 12, 1908. Standard specials continue to be manufactured in accordance with these specifications, but centrifugally cast pipes are now more extensively used.

The two methods used in casting pipe by the centrifugal process are (1) in a sand mold, and (2) in a water-cooled metal mold. In both methods, molten iron is fed at a uniform rate from a spout which is moved slowly into or out of the mold while the mold is revolving about its longitudinal axis in a horizontal position. A typical procedure in the manufacture of centrifugally cast iron pipe is as follows: Sand is rammed between the pattern and the mold while both are slowly revolving with their longitudinal axes in a vertical position. The pattern is withdrawn, and the mold is faced. The mold is then dried in the drying station. From here it proceeds to the casting machine where it is rapidly spun about its longitudinal axis in a horizontal position. The exact amount of metal required for the walls of the pipe is carefully weighed and poured into the revolving mold. The rate of revolution is usually such as to produce a force of 75*g* on the metal, *i.e.*, a force of 75 lb. for each pound weight of metal. This calls for a speed of about 1,550 r.p.m. for 3-in. pipe and 350 r.p.m. for 48-in. pipe. The molds are spun until the metal solidifies; then they are taken to the cooling room to cool slowly to assure annealing. Figure 135 shows the casting stage in a large cast-iron pipe foundry. After cooling, the flasks are stripped and the pipes are cleaned, tested, and coated.

FIG. 135.—Foundry of American Cast Iron Pipe Co. for the centrifugal casting of iron pipes. (*Courtesy of American Cast Iron Pipe Co.*)

Centrifugally cast pipe are commonly manufactured in accordance with Federal Specifications WW-P-421, with subsequent revisions.

296. Thickness of Cast-iron Pipe. Cast-iron pipe is manufactured in many different thicknesses. The thickness to be selected depends upon the forces to be resisted by the pipe. The following procedure in the determination of pipe-wall thickness is taken mainly from "Manual for the Computation of Strength and Thickness of Cast Iron Pipe," approved by A.S.A., December, 1939.[1]

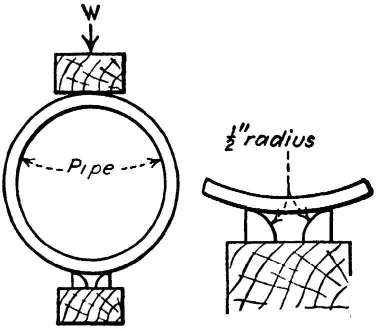

FIG. 136 — Three-edge bearing test for cast-iron pipe.

The load that will just cause breakage of a pipe is

$$w = \frac{W}{\sqrt{P}} \sqrt{P - p} \tag{1}$$

where w = external load per unit length
p = intensity of internal bursting pressure
W = external load per unit length that will crush pipe
P = intensity of internal pressure that will burst pipe

For a pipe supported on a three-edge bearing, illustrated in Fig. 136,

$$W = \frac{t^2 R}{0.0795(d + t)} \tag{2}$$

where t = net thickness of pipe wall, in.
d = internal diameter of pipe, in.
R = modulus of rupture, p.s.i , or metal in pipe, as determined by crushing tests, usually taken as 31,000

Values of the ratio W'/W are given in Table 53, in which W' is external load per unit length that will crush the pipe under the field conditions shown and W is the corresponding load under three-point bearing.

Net wall thickness is computed to resist external loads, as discussed in Sec. 191. Gross thickness is computed as the sum of net thickness to care for all loads, with allowance for foundry tolerance, corrosion, beam conditions, and a factor of safety, usually about 2.5. The allowance for beam conditions is particularly important for small pipes.

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[1] *Jour. A.W.W.A.*, December, 1939.

The empirical procedure in the determination of net thickness is as follows:

1. Assume two thicknesses, one slightly larger and the other slightly smaller than the true thickness, and compute the corresponding values of W and P from formula (2) and from the expression

$$P = \frac{2ts}{d} \tag{3}$$

where s = strength of iron in bursting tension, p.s.i., usually taken at 11,000.

Table 53.—Ratio of Trench Load to Equivalent Three-edge Bearing Load

Field condition[1]	Pipe diameter, in.															
	4	6	8	10	12	14	16	18	20	24	30	36	42	48	54	60
A	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15
B	1.29	1.32	1.34	1.36	1.38	1.41	1.43	1.45	1.47	1.52	1.58	1.64	1.69	1.72	1.75	1.77
C	0.22	0.31	0.40	0.50	0.60	0.67	0.73	0.78	0.81	0.87	0.93	0.96	0.98	0.99	0.99	1.00
D	0.82	0.83	0.84	0.86	0.88	0.91	0.95	0.98	1.01	1.07	1.14	1.19	1.23	1.25	1.28	1.31
E	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
F	1.75	1.78	1.80	1.83	1.85	1.88	1.90	1.93	1.95	2.00	2.08	2.14	2.20	2.25	2.29	2.31

[1] See following descriptions:
A = flat-bottom trench, not tamped.
B = flat-bottom trench, tamped.
C = pipe on blocks, not tamped.
D = pipe on blocks, tamped.
E = bottom of trench shaped to 90 deg. of pipe, not tamped.
F = bottom of trench shaped to 90 deg. of pipe, tamped.

2. With the computed values of W and of P plot a graph for expression (1), as a straight line on specially prepared coordinate paper with the abscissas for P plotted to a natural scale and with the ordinates for W varying as W^2. Such a plot is shown in Fig. 137.

3. The net value of t is found by interpolation, using the values of p and of w to be resisted.

The following is an example of the procedure:

Example: Let it be desired to determine the thickness of a cast-iron pipe for the following conditions: d = 48 in.; s = 11,000 p.s.i.; R = 31,000 p.s.i.; water hammer allowance = 70 p.s.i.; static pressure = 100 p.s.i.; external load due to backfill = 3,200 lb. per ft.; external load due to trucks = 1,400 lb. per ft., to be neglected in this example; factor of safety = 2.5; and field condition B, Table 53, prevails.

Solution: The three-edge bearing load equivalent to fill load plus truck load, with value 1.72, from Table 53, = 3,200/1.72 = 1,860 lb. per ft. length of pipe.

Hence

$$w = 1{,}860 \times 2.5 = 4{,}650$$
$$p = (100 + 70)2.5 = 425$$

Now assume that $t_1 = 1.10$ and $t_2 = 1.20$

Then

$$P = \frac{2ts}{d} = \frac{2 \times 1.1 \times 11{,}000}{48} \quad \text{or} \quad \frac{2 \times 1.20 \times 11{,}000}{48}$$

$$= 504 \text{ or } 550$$

$$W = \frac{t^2R}{0.0795(d + t)} = \frac{1.10^2 \times 31{,}000}{0.0795 \times 49.1} \quad \text{or} \quad \frac{1.20^2 \times 31{,}000}{0.0795 \times 49.2} = 9{,}610 \quad \text{or} \quad 11{,}420$$

The curves, or graphs, for these values of P and of W are shown in Fig. 137. By interpolation, using $p = 425$ and $w = 4{,}650$, the value of $t = 1.15$ in. is found. This value of t must be increased by the necessary foundry tolerance and corrosion allowance. The maximum foundry tolerances allowed by standard specifications are as follows: for pipes 3 to 8 in. in diameter, 0.07 in.; 10 to 24 in. in diameter, 0.08 in.; and 30 to 60 in. in diameter, 0.10 in. Corrosion allowance is 0.08 in. No thickness should be made less than a certain minimum permitted for the particular diameter and kind of iron.

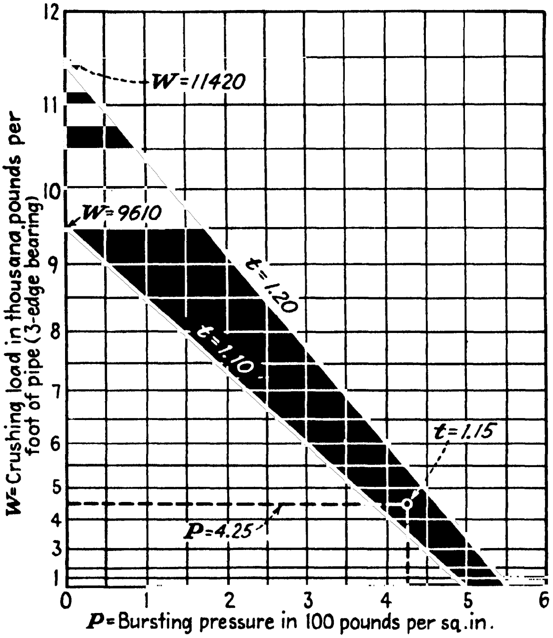


FIG. 137.—Determining thickness of cast-iron pipe by load pessure parabolas drawn as straight lines; 48-in. pipe used as sample. (*From Manual for the Computation of Strength and Thickness of Cast-iron Pipe, p.* 19, *Jour. A.W.W.A. December,* 1939.)

Calculated thicknesses are not used exactly, but the standard thickness, or the next thicker standard thickness, is used. A few standard thicknesses for some of the 14 classes of pit-cast pipe are shown in Table 54.

Centrifugally cast pipes, under Federal Specifications WW-P-421, for equal thicknesses are stronger than pit-cast pipes, since the permissible value of R, the modulus of rupture, is 40,000 p.s.i. Some standard thicknesses for pit-cast and centrifugally cast pipe are shown in Table 55.

TABLE 54.—SOME STANDARD THICKNESSES FOR PIT-CAST IRON PIPE[1]
Thickness in inches

Diam. in.	Standard thickness classes							
	1	2	4	6	8	10	12	14
4	0.40	0.43	0.50	0.58	0.68	0.79	0.92	1.07
8	0.46	0.50	0.58	0.68	0.79	0.92	1.07	1.25
12	0.54	0.58	0.68	0.79	0.92	1.07	1.25	1.46
18	0.63	0.68	0.79	0.92	1.07	1.25	1.46	1.71
24	0.74	0.80	0.93	1.08	1 26	1.47	1.72	2.01
36	0.97	1.05	1.22	1.43	1.66	1.93	2.25	2.62
48	1.18	1.27	1.48	1.73	2.02	2.35	2.74	3.20
60	1.39	1.50	1.75	2.04	2.38	2.78	3.24	3.78

[1] *Jour. A.W.W.A.*, December, 1939, Manual for Computation of Thickness, p. 3.

TABLE 55.—SOME STANDARD THICKNESSES FOR CENTRIFUGALLY CAST[1] AND PIT-CAST[2] IRON PIPE

Inside diam., in.	Centrifugally cast				Pit cast			
	Thickness, in.		Pounds per 12 ft. length		Thickness, in.		Pounds per 12 ft length with bell	
	150 p.s.i.	250 p.s.i.	150 p.s.i.	250 p.s.i.	130 p.s.i.	260 p.s.i.	130 p.s.i.	260 p.s.i.
4	0.34	0.38	195	220	0.48	....	280	
6	0.37	0.43	315	350	0.51	0.61	430	531
8	0.42	0.50	475	545	0.56	0.71	625	802
10	0.47	0.57	640	760	0.62	0.80	850	1,114
12	0.50	0.62	810	990	0.68	0.89	1,100	1,474
14	0.55	0.69	1,060	1,320	0.74	0.99	1,400	1,905
16	0.60	0.75	1,320	1,635	0.80	1.08	1,725	2,358
18	0.65	0.83	1,595	2,015	0.87	1.17	2,100	2,872
20	0.68	0.88	1,860	2,365	0.92	1.27	2,500	3,448
24	0.76	1.00	2,480	3,200	1.04	1.45	3,350	4,707

[1] Federal Specifications WW-P-421.

[2] American Water Works Association, 1908.

Class 150, centrifugally cast pipe, is frequently used in place of Class B (130 p.s.i.) for pit-cast pipe. Classes 100, 150, 200, and 250 represent working internal pressures in pounds per square inch under which *centrifugally cast pipes are designed to operate.*

297. ***Lengths of Cast-iron Pipes.*** Pit-cast iron pipe is manufactured *in standard lengths of 12 ft., 16 ft., and 5 m. Centrifugally cast pipe is available in standard lengths of 12, 16, 16½, 18, and 20 ft.* Advantages in the use of greater lengths is the smaller number of joints to be made and

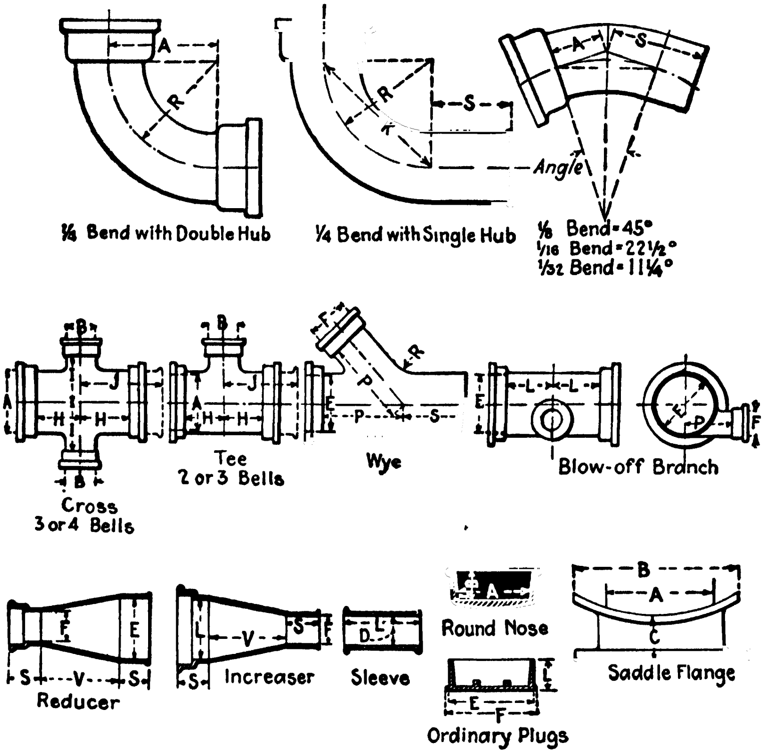


FIG. 138.—Standard cast-iron specials for water-pipe bell-and-spigot ends.

the smaller amount of leakage to be expected. Shorter lengths are, however, lighter and easier to handle. The cutting of cast-iron pipes to a desired length in the field is slow, difficult, and endangers the pipe because of cracking in the cutting process.

298. Cast-iron Pipe Fittings. The types and dimensions of bell-and-spigot fittings for cast-iron water pipes standardized by the American Water Works Association in 1908 and by the New England Water Works Association and the American Society for Testing Materials, which are in general use today, are illustrated in Fig. 138. The various branches shown and certain other specials are cast with all ends bell or with one end

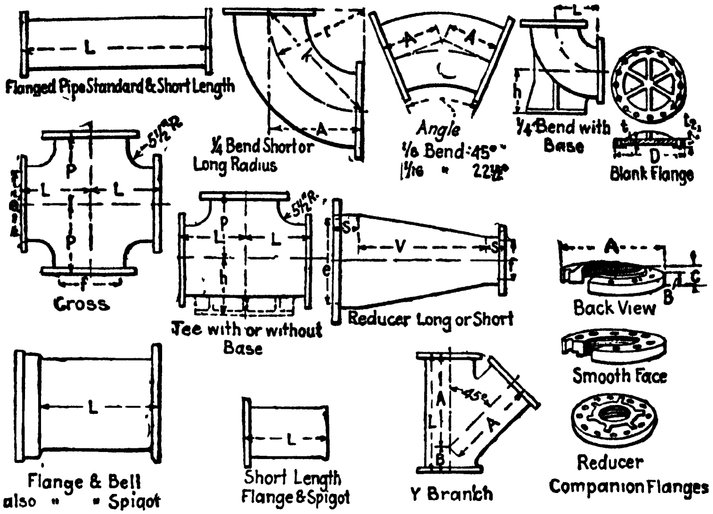


Fig. 139.—Standard specials with flanged ends for cast-iron water pipe.

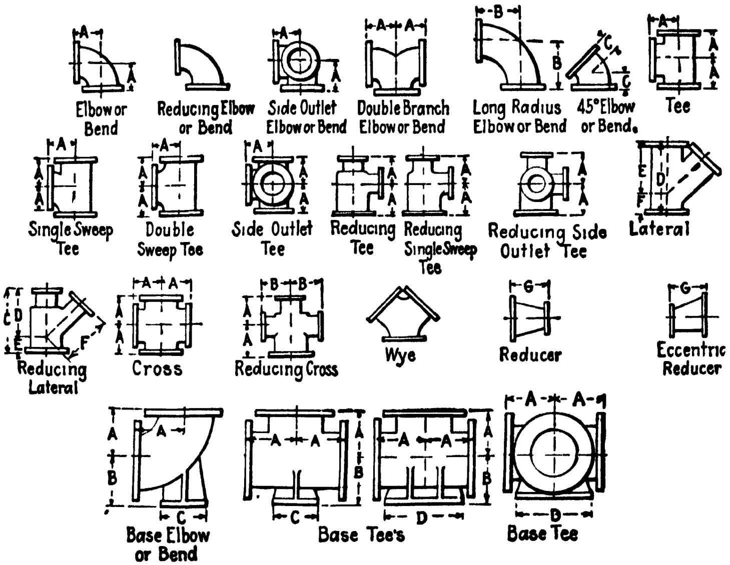
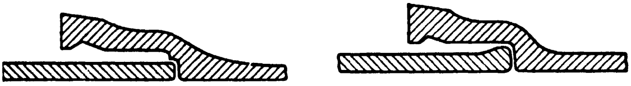


Fig. 140.—American standard fittings, flanged, for cast-iron pipe.

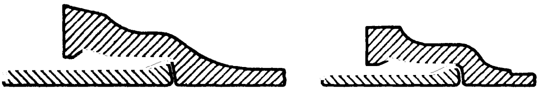
bell and the other end spigot. The casting of fittings not included among those considered "standard" is expensive. It is to be avoided where the use of standard specials can be devised in a satisfactory manner.

Standards for flanged cast-iron water pipe and fittings have not been adopted by any recognized waterworks authority. "Manufacturers' standards" are used in practice. They differ, more or less, among manufacturers. Some of the more commonly used flanged fittings are illustrated in Fig. 139. The American standard flanged fittings for steam pipe[1] give smaller radii and permit work in more confined space than the manufacturers' standards for waterworks fittings. Standard steam fittings are used in waterworks practice when the occasion demands. Some types of flanged fittings for steam are shown in Fig. 140.



CENTRIFUGALLY CAST TYPE I PIPE

CENTRIFUGALLY CAST TYPE II, PIPE



CENTRIFUGALLY CAST TYPE III PIPE

PIT CAST A.W.W.A. 1908 SPECIFICATIONS

Fig. 141.—Bell-and-spigot cast-iron pipe joints. Centrifugally cast pipe by Federal Specifications WW-P-421, 1931.

299. Joints for Cast-iron Pipe. Joints frequently used for cast-iron water pipe include bell-and-spigot, mechanical, flanged, and threaded. Other types are used under special conditions.

Bell-and-spigot joints are used principally for buried pipes. Sections through typical joints are shown in Fig. 141. The bell-and-spigot joint is made by slipping the spigot end of the pipe into the bell end of the adjacent pipe until contact is made at the base of the bell. If a lead joint is to be made, the space between the bell and the spigot is packed with hemp to a depth of 2 or 2½ in. The remaining space is filled with molten lead or lead substitute. In making a bell-and-spigot joint it is not necessary for the pipes to fit with exactness in order to hold the joint tight, and a slight variation in alignment is permissible, so that the pipe line can be laid around a long-radius curve. The joints require some skill in finishing, and they do not make a neat appearance when compared with some other types of joints.

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[1] Adopted by American Society of Mechanical Engineers, March 20, 1914.

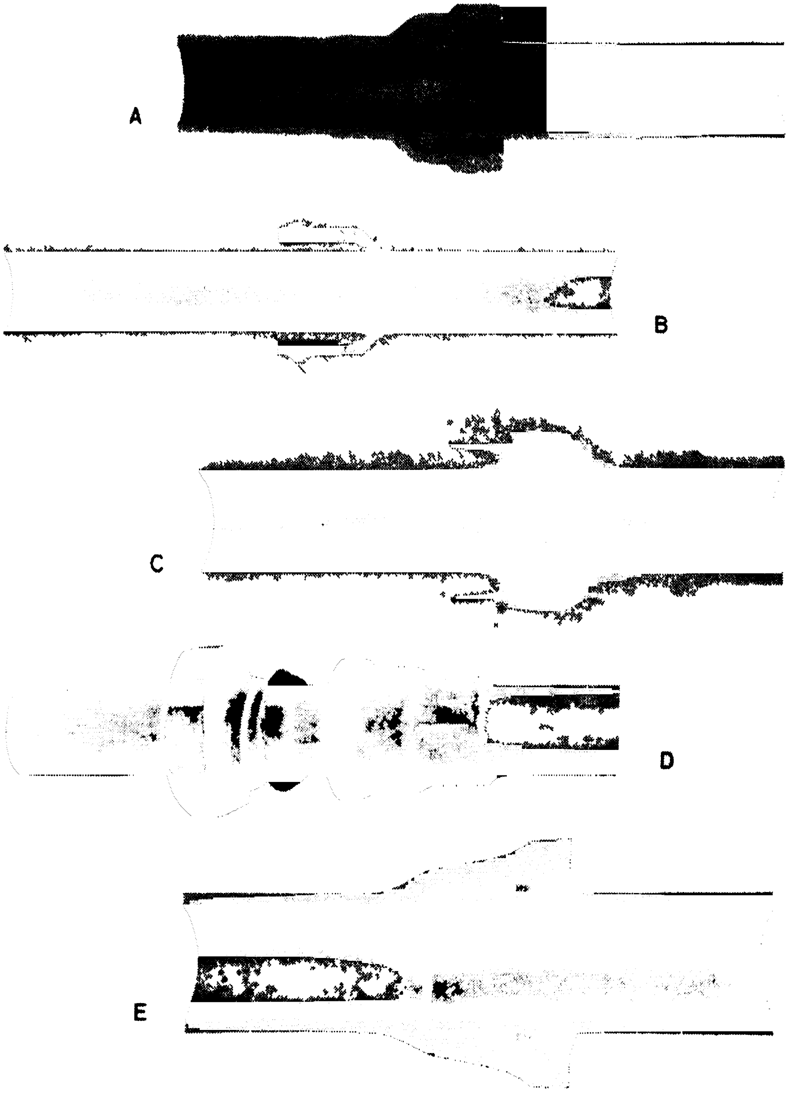


FIG 142 Types of cast iron pipe joints (A) Open bell pipe (bell and spigot without prepared joint) (B) prepared joint pipe (C) threaded intermediate joint (oversize threads) (D) mechanical joint pipe, (E) roll-on-rubber-ring joint pipe (*Courtesy of American Cast Iron Pipe Co*)

Mechanical joints and rubber-packed or filled joints have been more commonly used during and since the Second World War, primarily because of the scarcity of lead. Types of mechanical joints are shown in Figs. 142 and 143. A rubber-gasket joint is shown in Fig. 144. The sleeve-type coupling shown in Fig. 145, sometimes called a Dresser coupling, is a mechanical joint frequently used under unusual conditions. The complete joint consists of the sleeve, two gasket rings, and two following rings connected by bolts. The sleeve is slipped over or around the plain-end pipes, and the rubber packing rings are pushed home by tightening the bolts. The slipping of the coupling or joint on the pipe, as a result of expansion and contraction, can be prevented by placing a middle lug, cast on the sleeve, between the ends of the pipes at the joint. A satisfactory mechanical joint has a wedge-shaped rubber gasket held in place by a cast-iron follower which is drawn up and held in place by bolts fastened to lugs or rings on the end of the pipe corresponding to the bell end. In these gland-packed joints the rubber is held firmly under compression—a requisite for long service. Sleeve and gland-packed joints are flexible, watertight, durable, and do not require special skill or tools.

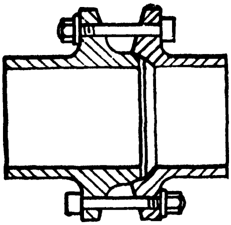


FIG. 143.—Universal pipe joint.

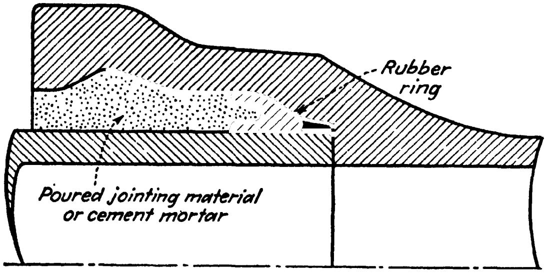
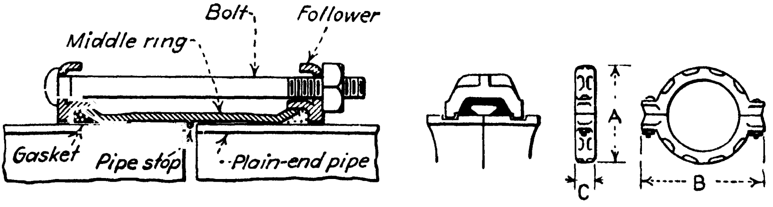


FIG. 144.—Rubber ring joint.

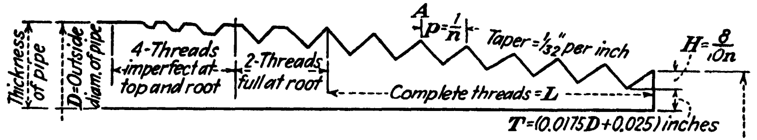
Flanged joints are used extensively on pipes within pumping stations, filter plants, and other close quarters. They require close fitting and perfect alignment. The joints are neat in appearance, rigid, and are easily made if the pipes are n alignment. Threaded joints are rarely used on cast-iron water pipes and never on pipes over 12 in. in diameter. They are relatively costly and offer no general advantages over flanged joints. Standard companion flanges are illustrated in Fig. 139, and American Standard pipe threads in Fig. 146. These standard threads were devised for wrought-iron steam pipe, but they have been adopted for use on cast iron, brass, copper, and other materials capable of being threaded.

Other types of joints used on cast-iron pipes include an expansion joint, a simple type of which is shown in Fig. 147; and a flexible joint, such as is illustrated in Fig. 148. The expansion joint is used on pipes exposed to appreciable differences of temperature. Such joints are not commonly used on buried cast-iron pipe carrying cold water. Flexible joints are used for pipes to be laid submerged under large bodies of water where the bottom of the trench or of the body of water may be uneven, around bends, and under conditions where the rigidity of nonflexible joints would preclude the laying of pipe.



Dresser coupling Victaulic coupling

FIG. 145.—Types of couplings for steel pipe.



n = Threads per inch

A = Diameter of thread at end of pipe $= D-(0.05D+1.1)P$

L = Length of completed threads, inches $= \frac{4.8+0.8D}{n}$

FIG. 146.—Standard pipe threads.

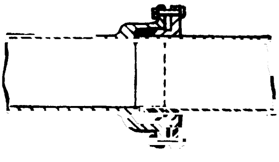


FIG. 147.—Expansion joint.

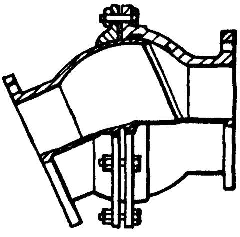


FIG. 148.—Flexible joint.

Most joints, except machined joints, require some material to fill the spaces in the joint to assure watertightness. Materials may be packed, calked, or poured into joints. In a packed joint the material is pushed into the joint and is held in place by a follower ring and bolts. Calking comprises the pounding of a compressible material into the spaces in the joint in which it is held by friction and shear. Poured joints are formed by pouring molten material into the joint where it cools and solidifies.

Poured lead joints must be subsequently calked owing to shrinkage of lead on cooling.

300. Joint Materials. Desirable features in joint compounds[1] include elasticity, strength, durability, adhesiveness, workability, availability, and economy; if the material is to be poured, no special skill is required in pouring. Materials used for joints in water pipes include lead, sulphur and sand mixtures, cement, rubber, and various proprietary substances with asbestos, hemp, jute, and other substances used as packing in poured joints.

The use of rubber for jointing materials is not uncommon. As a joint-filling substance rubber possesses many of the desirable features of an ideal joint material. When used in place of hemp a rubber packing ring will harbor no bacteria and, once sterilized, is less easily contaminated than hemp or jute. Rubber packing makes watertight joints, expands and contracts but little with temperature changes, is easy to use, and produces a flexible joint. Among its disadvantages, however, is its deterioration when exposed to gasoline or other petroleum products and its susceptibility to microbial attack.[2] Among recent types of elastic joints is one that utilizes round rubber rings deformed and adjusted in the joint as the spigot is pushed into the bell.[3]

Other joint materials, some of which may be proprietary, include Sinterite,[4] a war-developed substitute made by heating iron powder or iron oxides to form a spongy mass that can be calked like lead; and Fibrex, a sanitary paper packing with twisted pulp core and braided jacket.

301. Poured Joints. Lead has been considered to be the standard material for filling the space in a bell-and-spigot joint, but so many satisfactory and economical substitutes are available that its use for this purpose is diminishing. An important objection to lead is its lack of elasticity and its tendency to work loose under vibration. Lead substitutes include Leadite, Hydrotite, lead wool, Metalium, Tegul, and others.[5]

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[1] See PAYNE, C. R., "Jointing Compounds," *Jour. Penna. Water Works Operators' Assoc.*, Vol. 8, p. 85, 1936.

[2] See also ZoBELL, C. E., and J. D. BECKWITH, "The Deterioration of Rubber Products by Micro-organisms," *Jour. A.W.W A.*, April, 1944, p. 439

[3] See also HYDE, R. H., "The Hyde-Ro Ring," *Water Works & Sewerage*, September, 1940, p. 401.

[4] See also VOGT, HANS, *Gesundh.-Ing.*, Vol. 59, p. 631, abstracted in *Jour. A.W.W.A.*, June, 1939, p. 1071.

[5] THOMAS, R. J., "Lead Wool," *Jour. A.W.W.A.*, Vol. 5, p. 10, 1918; HEALEY, THOMAS, "Substitutes for Lead Joints," *Jour. A.W.W.A.*, Vol. 7, p. 234, 1920; KNOWLES, H. V., "Metalium," *Jour. A W.W.A.*, Vol. 5, p. 174, 1918; and PAYNE, C. R., "Tegul," *Water Works & Sewerage*, September, 1935, p. 317.

Lead wool is shredded lead. It is packed into the joints in strands about $\frac{1}{2}$ in. in diameter and 1 to 3 ft. long, depending on the size of the pipe. Calking is best done with a pneumatic tool, using a calking iron of the type shown in Fig. 149. Suphur compounds which include such proprietary products as Leadite, Tegul, Hydrotite, and Metalium, are composed principally of sulphur mixed with such substances as iron, slag, and salt. They are lighter than lead and possess some advantages over it. Sulphur and sand compounds melt at about 240°F., become liquid at 248°F., and ignite at 280°F.

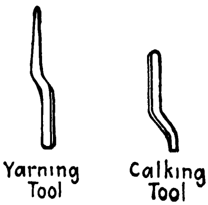

FIG. 149.—Calking tools.

Sulphur-compound joints tend to become tighter with time, whereas where lead joints leak the condition tends to become progressively worse. Specifications for newly laid pipe may provide that the leakage should not exceed 100 gal. per day per mile of pipe per inch nominal diameter for pipe in 12-ft. lengths, 75 gal. for 16-ft. lengths, and corresponding amounts for other lengths of pipe.[1] The test for leakage is to be made under a pressure of 150 p.s.i. "Tests made immediately after the completion of a pipe laid with sulphur compound joints showing a relatively high rate of leakage are not at all indicative of the ultimate tightness of that line . . . (because) sulphur-compound joints will seep in the early stages of their life, but will take-up as they age."[1]

Cement is used successfully for bell-and-spigot joints in both concrete pipes and metal pipes. The material may be either packed into place as a stiff mortar or poured into the joint space as cement grout.[2]

302. Wrought Pipe and Fittings. There is some confusion about the quality of materials known as wrought iron and that of wrought steel for pipe. This is due partly to the similarity of methods of manufacture and of the quality of the materials in the two kinds of pipe. The term "wrought pipe" partly overcomes the difficulty. Wrought pipe is manufactured by rolling flat plates of the metal to the proper diameter and welding the edges. It is used principally for installations within buildings where it is protected against the agents of corrosion that are more active in exposed locations. The material is used in pipes from ⅛ to about 20 in. in diameter. Compared with cast iron, the pipe is lighter; is more easily cut, threaded, and worked; and, for interior work, presents a neater appearance. It is less durable and is more expensive than cast-iron pipe.

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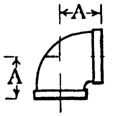
[1] "Tentative Standard Specifications for Laying Cast-iron Pipe," *Jour A.W.W.A*, February, 1938 p. 215.

[2] See also PRACY, G. W., "Cement Joints for Cast Iron Water Pipe," *Jour. A.W W.A*, Vol 7, p. 436, 1920; SHAW, C H, "Cement Joints for Cast Iron Water Mains," *Proc A S.C.E.*, Vol. 43, pp. 433, 1303, 1917; ALEXANDER J L, and R M. EBAUGH, "Cement Joints for Improved Water Quality," *Jour. A W.W A*, April, 1941, p. 769.

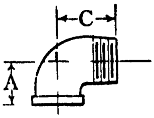
Couplings and nipples are the only fittings made of wrought iron or steel. Other ferrous metal fittings are made of cast or malleable iron.

TABLE 56.—DIMENSIONS OF STANDARD MALLEABLE IRON FITTINGS
(Crane Company)

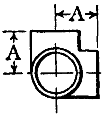
Dimensions, in Inches



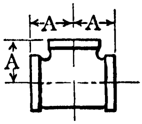
90° Elbow



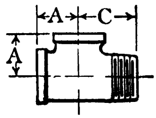
90° Street Elbow



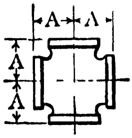
Side Outlet Elbow



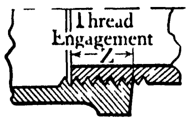
Tee



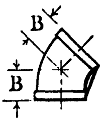
Service Tee



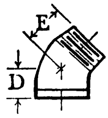
Cross



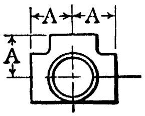
Box Union



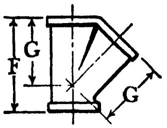
45° Elbow



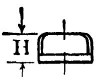
45° Street Elbow



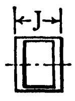
Four-Way Tee



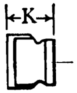
45° Y Bend



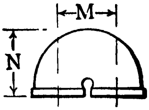
Cap



Coupling



Reducer



Return Bend

Size	A	B	C	D	E	F	G	H	J	K	L	Return bends								
⅛	11/16	½	1	11/16	11/16			9/16	15/16		¼	Size	M	N	Size	M	N	Size	M	N
¼	13/16	¾	1 1/16	⅝	15/16			⅝	1 1/16	1	⅜	Close pattern			Open pattern			Wide pattern		
⅜	15/16	13/16	1 7/16	13/16	1 1/16	2⅛	1 7/16	¾	1 3/16	1⅛	⅜	½	1	1¾	½	1½	1⅞	⅜	1½	1⅝
½	1⅛	⅞	1⅝	13/16	1 3/16	2 7/16	1 11/16	⅞	1 5/16	1¼	½	¾	1¼	2 3/16	¾	2	2¼	¾	4	3 3/16
¾	1 5/16	1	1⅞	15/16	1 5/16	2 13/16	2 1/16	1 1/16	1½	1 7/16	9/16	1	1½	2½	1	2½	2⅝	¾	6	4½
1	1½	1⅛	2⅛	1 1/16	1½	3⅛	2 7/16	1 3/16	1 11/16	1 11/16	11/16	1¼	1¾	2 13/16	1¼	3	3 1/16	1	6	4½
1¼	1¾	1 5/16	2 7/16	1¼	1 11/16	4 1/16	2 15/16	1¼	1 15/16	2 1/16	11/16	1½	2 3/16	3 3/16	1½	3½	3⅝	1¼	6	4¾
1½	1 15/16	1 7/16	2 11/16	1⅜	1⅞	4½	3 5/16	1 5/16	2⅛	2 5/16	11/16	2	2⅝	3⅞	2	4	4⅜	1½	6	5
2	2¼	1 11/16	3¼	1 11/16	2¼	5 7/16	4	1 7/16	2½	2 13/16	¾	Medium pattern			2½	4½	4 15/16	2	5	4¾
2½	2 11/16	1 15/16	3 13/16	1⅞	2 9/16	6¼	4 11/16	1 11/16	2⅞	3¼	15/16	½	1¼	1⅝	3	5	5 9/16	2	6	5 5/16
3	3⅛	2 3/16	4½	2⅛	3	7¼	5 9/16	1 13/16	3 3/16	3 11/16	1	¾	1½	1 15/16	4	6	6 11/16	3	7½	6 13/16
3½	3 7/16	2⅜	5⅛	2⅜	3⅜			1 15/16	3 7/16	4	1 1/16	1	1⅞	2¼				3	8	6 13/16
4	3¾	2⅝	5 11/16	2½	3¾	8⅞	6 15/16	2 1/16	3 11/16	4⅜	1⅛	1¼	2¼	2 13/16				6	12	11 3/16
5	4½	3 1/16	6⅞					2 5/16	4¼	4⅛	1¼	1½	2½	3 3/16						
6	5⅛	3 7/16	8					2 9/16	4¾	4⅜	1 5/16	2	3	3⅞						

Malleable iron is cast iron of such quality that it has some toughness which permits its being bent or pounded to some extent. It cannot be forged, and it can be broken by a hammer blow. Some dimensions of standard fittings are shown in Table 56.

303. Steel Pipe. Specifications for steel pipe of various sizes and for various applications have been prepared. References to most of these specifications are given in Table 52. Seamless steel tubing or mill pipe is made in sizes from ⅛ to 24 in. Thicknesses, weights, and working pressures for nine sizes of pipe are shown in Table 57. Pipes made of rolled steel are used mainly on long pipe lines with diameters greater than 3 or 4 ft. They are particularly suitable on bridges or under similar conditions where strength and low weight are desired. Steel pipe cannot be easily made to resist high external pressure. Hence, a steel pipe should be placed under favorable conditions in which high external loads are not expected to fall on the pipe unless the pipe is corrugated or otherwise reinforced. Since buried steel pipe may deteriorate rapidly, it should be exposed and accessible to inspection and repair.

TABLE 57.—SOME DIMENSIONS OF STEEL PIPE FOR UNDERGROUND WATER SERVICE

Nominal diam., in.	Outside diam., in.	Wall thickness, in.	Weight per foot, pounds	Working pressure p.s.i.	Nominal diam., in.	Outside diam., in.	Wall thickness, in.	Weight per foot, pounds	Working pressure, p.s.i.
4	4.5	0.125	5.84	690	14	14 000	0 188	27 65	340
4	4.5	0 337	14.98	1,870	14	14.000	0.500	72 09	890
6	6.625	0.188	12.89	710	18	18 000	0.188	35 67	260
6	6.625	0 432	28 57	1,630	18	18.000	0.500	93.45	690
8	8.625	0 188	16 90	550	20	20 000	0.188	39.67	240
8	8.625	0.500	43.39	1,450	20	20.000	0.500	104.13	630
10	10 750	0.188	21.15	440	24	24.000	0.188	47.68	200
10	10.750	0.500	54 74	1,160	24	24.000	0.500	125 49	520
12	12.750	0.188	25.15	370					
12	12.750	0.500	65.42	980					

From "Standard Specifications American Water Works Association." See *Jour A.W.W.A.*, April 1943, p. 475.

There are eight or more thicknesses for each diameter between the greatest and the least thickness shown for each diameter in this table.

The life of steel pipe under ordinary conditions is greatly dependent on the quality of the protective coating. Steel pipes have been found in good condition after 50 years, and they have depreciated materially in 20 years.[1]

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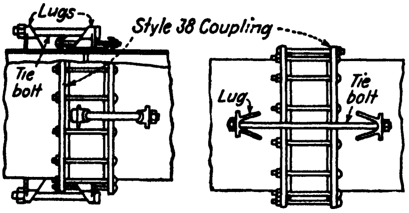
[1] HILL, N. S., discussion, *Proc., A.W.W.A.*, 1911, p. 227.

Joints in steel pipe[1] are made by riveting or welding. The joints may be either transverse and longitudinal, or transverse and spiral. Spiral-welded steel pipe is available in sizes from 6 to 36 in. in diameter and in lengths up to 40 ft. Couplings or joints of the Dresser type, illustrated in Fig. 145, and of the Victaulic type, shown in Figs. 145 and 150, depend on rubber rings or lips for watertightness; they provide flexibility not to be found in riveted or in welded joints. The couplings are prevented from working off the joint by a rib on the inside of the sleeve which goes between the ends of the pipes being connected. Joints for service connections for water consumers cannot be made by tapping, as they are in cast-iron pipe, unless the steel is ¼ in. or more in thickness. Pipe saddles, with the proper size hole tapped in them, may be clamped to the steel pipe, or a plate may be welded directly to the steel pipe and then drilled and tapped. Tees, crosses, ells, and other specials made of welded steel, with flanged ends, are available for working pressures of 125 p.s.i.



TIE-BOLTS WITH UNIVERSAL JOINT ACTION

TIE-BOLTS ACROSS JOINT

FIG. 150.—Mechanical joint harness to take longitudinal pull. (*From W. J. Hurlbut, Jour. A.W.W.A., October, 1943, p. 1284.*)

304. Design of Plate Steel Pipe. The plate thickness t, of a steel pipe, required to resist internal bursting pressure is given by the formula

$$t = \frac{pR}{f e}$$

where p = internal bursting pressure, p.s.i.
R = radius, in.
f_t = allowable unit stress, p.s.i
e = joint efficiency

The joint efficiency is taken as the smaller of the two values, efficiency of plate and efficiency of rivets. The efficiency of the plate is $e = (r - d)/r$ and of the rivets $e = (ns/rtf_t)$ in which r is the pitch and d is the diameter of the rivets. s is the smaller of the two values, shearing or bearing value, of the rivet, t is the plate thickness, and n is the number of rivets in a lap joint or on either side of a double-strap butt joint. Welded pipe is ordinarily considered to have a joint efficiency of unity.

Longitudinal stresses are not so important in buried pipes but are considered in uncovered pipes. In the latter case longitudinal forces may occur due to (1) the inclination of the pipe from the horizontal, (2) changes in length due to temperature which tend to cause the pipe to develop frictional resistance at the supports, (3) the frictional resistance of expansion joints, and (4) pressures acting on reducers in the pipe line. The unit stresses f_c or f_t in the pipe shell may be determined by the formulas

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[1] See also, BARNARD, R. E., "Fittings and Specials for Steel Pipe," *Jour. A.W.W.A.*, October, 1941, p. 1751.

$$\text{Compression, } f_c = \frac{\Sigma F}{a} \qquad \text{or} \qquad \text{tension, } f_t = \frac{\Sigma F}{ae} \tag{4}$$

where ΣF = summation of longitudinal loads
a = cross-sectional area of pipe shell
e = efficiency of girth joints

Stress intensities due to changes in temperature computed from formula (3) in Chap. X may also be considered.

The design of plate-steel pipe to resist combined direct and flexural stresses is a specialized problem, particularly for pipes having diameters greater than about 3 ft. The magnitude of the stresses is governed by the loads and conditions stated in Sec. 186 and by the manner in which the pipe is supported and stiffened. Design methods may be found in standard books and professional papers.[1]

The critical external pressure that will cause buckling in a pipe without stiffening rings is given approximately by the formula[2]

$$P_{cr} = 20\frac{EI}{d^3} = \frac{5E(t)^3}{3(d)} \tag{5}$$

where P_{cr} = external pressure per unit of length
E = modulus of elasticity of the metal
I = moment of inertia ($t^3/12$ for a unit length)
t = thickness of pipe
d = diameter of pipe

305. Wood Pipe.[3] Wood-stave pipes are of two types: continuous stave, as illustrated in Fig. 151, and machine-banded, as illustrated in

---

[1] Report of Hydraulic Power Committee, National Electric Light Association, 1923. CREAGER, W. P., and J. JUSTIN, "Hydroelectric Handbook," Chap. XX, John Wiley & Sons, Inc., New York, 1927; PETER, BIER, *Eng. News-Record*, Vol. 99, p. 629, 1927; SCHORER, H., "Design of Large Pipe Lines," *Trans. A.S.C.E.*, Vol. 98, p. 101, 1933. SCHOKLITSCH, A., "Hydraulic Structures," trans. by Samuel Shulits, A.S.M.E., 1937.

[2] *Univ. Ill. Expt. Sta., Bull.* 5, 1906.

[3] See also ADAMS, A. L., "Stave Pipe—Its Economic Design and the Economy of Its Use," *Trans. A.S.C.E.*, Vol. 41, p. 25, 1899; ADAMS, A. L., "Additional Information on the Durability of Wooden Stave Pipe," *Trans. A.S.C.E.*, Vol. 58, p. 65, 1907; LEDOUX, J. W., "Cast-iron and Wood-stave Pipe Economically Compared," *Eng. News-Record*, Vol. 85, p. 932, 1920; National Tank and Pipe Co., "Wood Pipe Handbook," 1938.

Fig. 152. They are made of any desired diameter, up to about 20 ft., and for internal pressures up to about 150 p.s.i., or even higher. Pine, fir, redwood, red cedar, and cypress are commonly used for making wood pipe. Among the advantages of the use of wood pipe may be included light weight, fairly long life under favorable conditions, high carrying capacity or value of C in Hazen and Williams formula, freedom from electrolysis or electryolytic current, and easy transportation when knocked down. It is not suitable for high external loads, or for conditions under which the wood is alternately wet and dry.

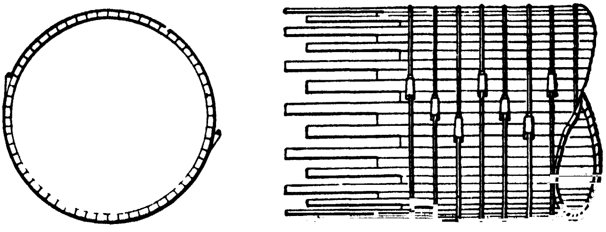
Cross Section Elevation

FIG. 151.—Continuous wood-stave pipe.

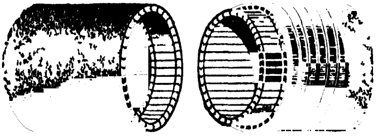
FIG. 152.—Machine-banded wood-stave pipe.

Continuous-stave pipe is usually manufactured in cradles at the point of installation. The staves are laid together in such a manner that the joints between the ends of the staves do not come at the same transverse section. The joints between the sides of the staves are planed radially. A copper or zinc plate is sometimes fitted between end joints to secure watertightness, because of the slight longitudinal expansion of wood on swelling due to moisture. The staves are held in place by wire bands. The ends of the wires are held firmly in cast-iron shoes, such as are shown in Fig. 153, and the bands are drawn tightly by turning the nuts, also shown in the figure. Continuous-stave pipe which is fabricated at the site of installation can be made of one continuous length, and the flexibility of the wood is such that the pipe can be laid on curves with a minimum radius about fifty times the diameter of the pipe.

Machine-banded pipes are usually made in the factory and are shipped to the site of installation. The staves are held together by spirally wound steel bands. The pipes are made in almost any length, up to 20 ft. Adjacent sections of pipes are joined by collars that are placed around the outside of the pipe or by means of recessed or tenon joints, as shown in

Fig. 154. The latter are suitable only for low pressures. The collars may be made of wood staves, cast iron, or steel. Where collars are used, it is necessary to leave an unbanded space at the end of each pipe over which the collar fits. The collars, when made of wood, are usually machine banded, and they should receive particular protection against rot, because they are not protected by the moisture in the pipe.

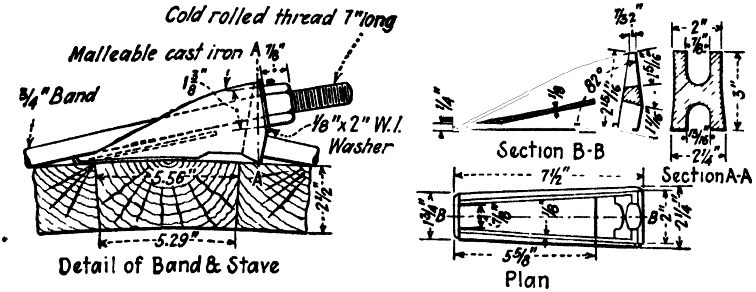

Fig. 153.—Cast-iron shoe for holding bands on wood-stave pipe.

The backfilling of wood-stave pipe should be done with great care, particular attention being taken to tamp the backfill firmly under and around the pipe to the height of the horizontal diameter. Failure to do this results in high leakage or failure of the pipe, owing to the weight of the backfill over the top of the pipe. Dry sand constitutes the best material for backfill next to the pipe. It must, however, be restrained. Loam and rich soil should not be placed in close proximity to wood pipe. If the pipe is not placed in a trench, it is supported by cradles at suitable intervals.

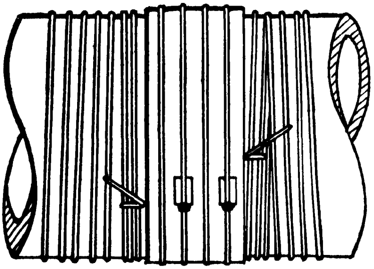

Fig. 154.—Joint for machine-banded wood-stave pipe.

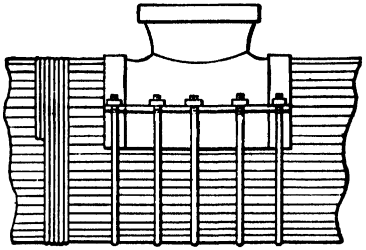

Fig. 155.—Typical cast-iron connection for wood-stave pipe.

The pipe should be slowly filled with water to allow the staves to swell and close leaks before the full water pressure is placed in it. Large leaks might cause erosion of the backfill and endanger the safety of the pipe.

Connections to wood pipe are made by special iron castings which are fastened to the wood pipe in a manner similar to that shown in Fig. 155.

306. Design of Wood Pipe. Wood-stave pipe is designed to resist internal bursting pressures. Flexural stresses are usually neglected in design, but their effects may cause failure of the pipe if proper measures to resist them are not provided for in construction and backfilling. Adequate tamping to the height of the horizontal diameter is essential for the safety of buried pipe.

In designing a wooden pipe made up of staves held together by metal bands, two factors are to be considered in the design of the bands: their diameter and the spacing between them. As these factors are dependent upon each other, one must be fixed before the other can be determined. It is customary to fix the diameter first, so as to develop the full crushing strength of the wood. Under such circumstances, the diameter of the band is independent of the internal bursting pressure. The distribution of stresses on a band around a wood-stave pipe is illustrated in Fig. 156. In this figure

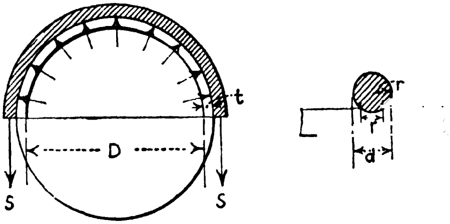

FIG. 156.—Stresses in bands on wood-stave pipe.

$$2S = (D + 2t)ae \tag{6}$$

where S = total stress in band
D = internal diameter of pipe
t = thickness of wood stave
e = crushing strength of wood across grain (sometimes taken as about 650 p.s.i.)
a = width of band in contact with wood
R = internal radius of pipe
r = radius of band
f_t = strength of steel in band in tension (sometimes taken between 15,000 and 20,000 p.s.i.)

It is common practice to assume that $a = r$ per unit length of the band. Then

$$r = \frac{(R + t)e}{\pi f_t} \tag{7}$$

The full strength of each band is $S = \pi r^2 f_t$, and this must equal the total internal pressure between the bands. The internal pressure per unit length of pipe is

$$2pR + 2tW \tag{8}$$

where p = intensity of internal bursting pressure
W = swelling strength of wood. This is often assumed to be about 100 p.s.i.

Then if B represents the distance between bands, *i.e.*, the pitch of the bands,

$$B(2pR + 2tW) = 2\pi r^2 f_t \tag{9}$$

Therefore

$$B = \frac{\pi r^2 f_t}{(pR + tW)} \tag{10}$$

It is possible to determine the diameter and spacing of steel bands on wood-stave pipe by the use of formulas (7) and (10). The maximum spacing between bands is dependent upon the strength of the staves to resist bending and the allowable deflection. A spacing greater than 10 to 12 in. is unusual. The minimum limit of band spacing is fixed by the size of the band shoe and the possibility of turning up the nut.

The assumption that, for circular bands, the width of the contact surface is equal to the radius of the band is approximate only. Actually, the smaller the band, the greater the relative area in contact.[1] Circular or oval bands are preferable to flat bands, as the former expose relatively less surface to the atmosphere to be corroded, and the condition of the bands can be more easily inspected. In general, the wood outlasts the bands, particularly if both are constantly wet.

307. Concrete Pipe. Concrete pipe is used on long conduits and aqueducts but is not commonly used in distribution systems. Among the advantages of concrete pipe may be included:

1. Suitability to resist loads due to backfilling and due to collapsing external pressures.
2. Low cost of maintenance.
3. The weight tends to decrease the liability of the pipe to float when empty.
4. Transportation of cement and reinforcing steel is required only when local aggregates and water can be used in making the pipe.
5. It is not subject to corrosion when buried in ordinary soil or when carrying normal, potable water.
6. Expansion joints are not normally required.
7. Specially skilled labor is not required for its construction.

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[1] See also HENNY, D. C., "Discussion of Wood Stave Pipe," *Trans. A.S.C.E.*, Vol. 41, p. 71, 1899.

Among the disadvantages may be included:

1. *Tendency to leak as a result of porosity and shrinkage cracks.*
2. *Corrosion in presence of alkali or acid.*
3. *Difficulty to repair.*

Concrete pipe may be poured in place, or it may be precast and shipped to the job. Because of better control of the concrete mix and tamping facilities afforded in the casting yard, precast pipe can be made thinner than monolithic pipe cast in place. For diameters up to 5 or 6 ft. precast pipe is less expensive and is of better quality than pipe cast in place. Monolithic pipe is desirable for diameters greater than 60 in., although precast pipe of larger diameters have been used. A saving in form costs and a better distribution of the foundation reaction may be had by the use of a section similar to that shown in Fig. 157, instead of the hollow-ring type shown in Figs. 158 and 159. Tentative Emergency Specifications for Reinforced Concrete Pressure Pipe, Approved by the

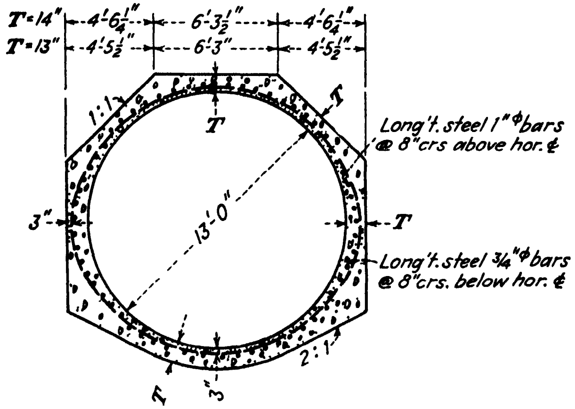

T=14"
4'-6¼"
6'-3½"
4'-6¼"
T=13"
4'-5½"
6'-3"
4'-5½"
1:1
T
T
Long't. steel 1"φ bars @ 8" crs above hor. ℄
3"
13'-0"
T
Long't. steel ¾"φ bars @ 8" crs. below hor. ℄
2:1
T
3"

FIG. 157.- Typical siphon barrel.

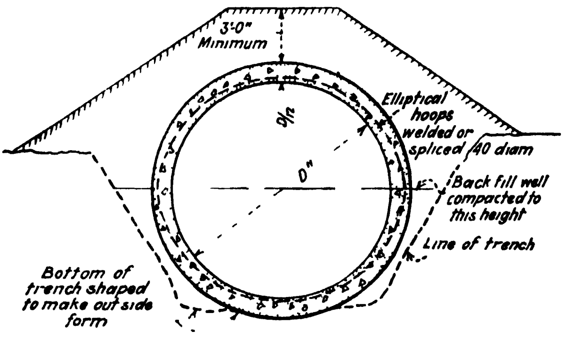
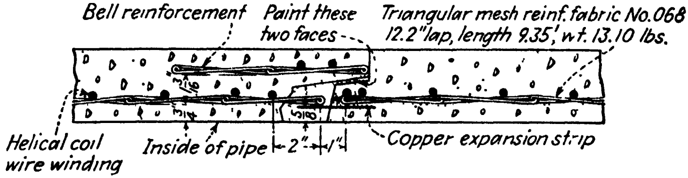

3'-0" Minimum
0"
Elliptical hoops welded or spliced 40 diam
Back fill well compacted to this height
Line of trench
Bottom of trench shaped to make outside form
T

FIG. 158 – Typical hollow-ring type of reinforced-concrete pipe.

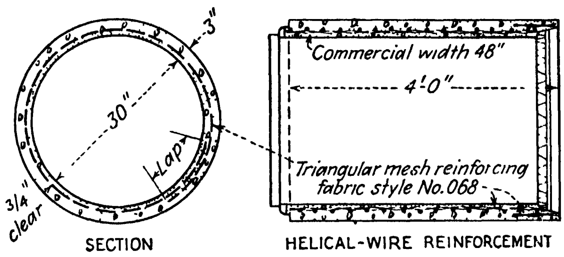
Board of Direction, American Water Works Association, March, 1943,[1] are intended for pipe to be used under normal conditions. They cover three types of concrete pipe, as shown in Table 58.


Bell reinforcement
Paint these two faces
Triangular mesh reinf. fabric No. 068 12.2" lap, length 9.35', wt. 13.10 lbs.
Helical coil wire winding
Inside of pipe
2"
1"
Copper expansion strip

HELICAL-WIRE REINFORCEMENT

DETAILS OF STANDARD PRESSURE PIPE

HEADS MORE THAN 100 FEET



SECTION HELICAL-WIRE REINFORCEMENT

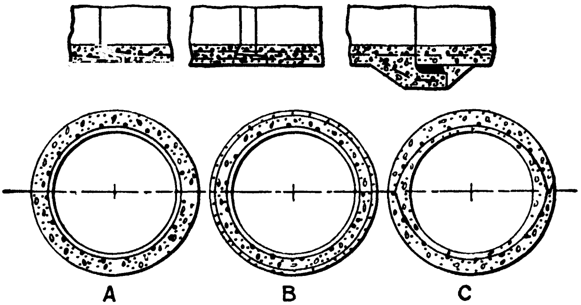


FIG. 159 - Methods for reinforcing concrete pipe.

Watertightness is obtained in low-pressure concrete pipe by providing sufficient thickness of good concrete and longitudinal reinforcement equal to at least 0.25 per cent of the cross-sectional area of the concrete and by reducing allowable unit stress in the steel as the internal pressure increases. For large-diameter pipes a minimum wall thickness of 1 in. for each foot in diameter of pipe is a good rule to follow in design. Surface washes may be applied to the concrete, after it has set, to assure watertightness.

Watertightness of high-pressure concrete pipe may be obtained by the insertion of a thin steel cylinder in the pipe walls either with or without prestressed reinforcement.[1] Lighter weight pipes may be used when the reinforcement is prestressed because the concrete is kept under compression at higher internal pressures than when not prestressed.[2]

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[1] See *Jour. A.W.W.A.*, April, 1943, p. 501.

TABLE 58.—WALL THICKNESS FOR REINFORCED-CONCRETE PRESSURE PIPE*
(The thicknesses given are minimum expressed in inches)

Diam. in.	Type			Diam. in.	Type			Diam. in.	Type		
	A	*B*	*C*		*A*	*B*	*C*		*A*	*B*	*C*
20	3⅛	3⅛	1¾	39	4	4	3½	60	5½	6	5
24	3½	3½	2½	42	4	4½	3¾	66	6	6½	5½
27	3½	3½	2⅝	46	4½	4½	x	72	6½	7	6
30	3½	3½	2¾	48	4½	5	4⅛	78	7	7½	
34	3½	3½	x	51	5	5	4¼	84	7½	8	
36	3½	4	3⅛	54	5	5½	4½				

* Approved March, 1943, by A.W.W A. See *Jour. A.W.W.A.*, April, 1943, p. [illegible]

Reinforcement may be placed in concrete pipe to enable it to withstand the shocks of handling and of external pressure. For ordinary conditions, reinforcement is placed in precast pipe, as experience has indicated will be satisfactory, no attempt being made to compute the indeterminate stresses that may be encountered after the pipe is in place. Methods for the reinforcing of concrete pipe are illustrated in Figs. 157, 158, 159, and 160. Reinforcement, as shown in Fig. 159*C*, is most efficient if the pipe is placed in the position shown. If, however, the pipe is turned 90 deg., the reinforcement is almost useless. In monolithic pipes of diameters over 60 in., elliptical hoops are usually used as reinforcement except for the higher internal pressures. These should be placed as shown in Fig. 158. Welded hoops have an advantage in that they eliminate laps and thus effect a saving of steel and permit a closer spacing of hoops where necessary. A double layer of circular hoops is used as reinforcement for resisting stress due to high heads.

Precast concrete is manufactured also under proprietary rights by various processes. Lock-joint pipe is a pipe that may be so manufactured either as precast or as cast-in-place pipe. This pipe receives its name from the type of joint used, details of which are shown in Fig. 159.[3]

[1] See also LONGLEY, F. F., "Prestressed Reinforced Concrete Pipe," *Jour. New Engl. Water Works Assoc.*, December, 1945, p. 335.

[2] See also "Prestressed Concrete Pipe Details are Explained," *Water Works Eng.*, Mar. 20, 1946, p. 291.

[3] LONGLEY, F. F., "Manufacture and Construction of 48-in. Lock-joint Pipe Line," *Jour. New Engl. Water Works Assoc.*, Vol. 49, p. 212, 1926.

Concrete pipe is subject to corrosion[1] when exposed to moist acid soils or to ground impregnated with magnesium or calcium sulphates. This corrosion is a result of bacterial activity, as explained in Sec. 339. Most failures due to corrosion are traceable to inferior materials or to poor workmanship.

Since the coefficient of expansion of concrete is nearly the same as that of steel, similar allowance for expansion and contraction must be made in each kind of pipe. Usually, however, concrete pipe is buried so that wide changes of temperature are not experienced, and more commonly, expansion and contraction is taken up in the joints in the concrete pipe.

308. Design of Reinforced-concrete Pipe. Reinforcing steel is placed in concrete water pipe to resist the tensile stresses due to bursting pressure and flexure. The concrete carries the compressive stresses in pipe subject to flexural stresses. When bursting pressure alone is considered, the area of steel per foot of pipe is computed by the formula

$$A_s = \frac{pr}{f_t} \times 12 \tag{11}$$

where p = internal pressure, p.s.i.
r = radius of pipe, in.
f_t = allowable tensile stress, p.s.i.

When flexural stresses are considered, the moments and tensions due to the loads causing bending are calculated for critical points by an elastic analysis or from known coefficients, such as are given in Table I of Fig. 160. To the tension thus determined is added that due to internal pressure. The section is then designed by usual methods for reinforced concrete for the combined stress due to tension and bending. Distribution of cracks that might cause leakage is controlled by the proper selection of bar sizes and allowable working stresses. Longitudinal reinforcement is usually used to distribute cracks due to temperature effects and contraction due to setting. The total area of longitudinal steel usually placed is ¼ to ½ per cent of the cross-sectional area of the concrete in the pipe.

309. Asbestos-cement Pipe. Asbestos-cement pipe is composed of asbestos fiber and portland cement combined under pressure into a dense homogeneous structure in which a strong bond is effected between the cement and the asbestos fiber. Among its advantages may be included high resistance to most corrosive conditions; freedom from electrolysis; light weight, being about 25 per cent lighter than standard pit-cast iron pipe; easily cut and fitted; easily handled; not easily broken in handling;

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[1] See also "Corrosion of Cement and Concrete," *Surveyor*, Mar. 2, 1945, p. 123.

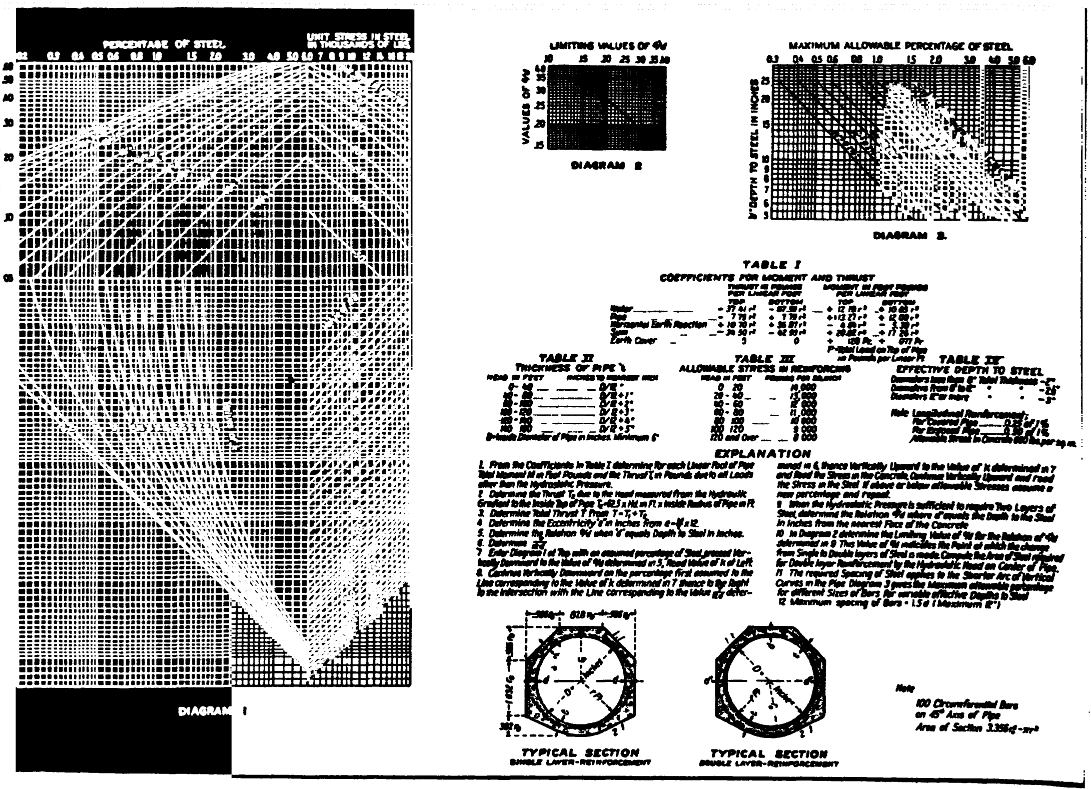

TABLE I

COEFFICIENTS FOR MOMENT AND THRUST

	THRUST IN POUNDS PER LINEAR FOOT		MOMENT IN FOOT POUNDS PER LINEAR FOOT	
	TOP	BOTTOM	TOP	BOTTOM
Water	$-37.41r^2$	$-87.59r^2$	$+12.19r^3$	$+10.65r^3$
Pipe	$-7.79r^2$	$+7.79r^2$	$+13.27r^3$	$+12.00r^3$
Horizontal Earth Reaction	$+10.70r^2$	$+36.87r^2$	$-4.64r^3$	$-3.39r^3$
Sum	$-34.50r^2$	$-62.93r^2$	$+20.82r^3$	$+17.26r^3$
Earth Cover	0	0	$+.108\ Pr$	$+.077\ Pr$

P = Total Load on Top of Pipe in Pounds per Linear Ft.

TABLE II

THICKNESS OF PIPE t

HEAD IN FEET	INCHES TO NEAREST INCH
0-40	D/12"
40-80	D/12 + 1"
80-100	D/12 + 2"
100-120	D/12 + 3"
120-140	D/12 + 4"
140-160	D/12 + 5"

D = Inside Diameter of Pipe in Inches. Minimum 6"

TABLE III

ALLOWABLE STRESS IN REINFORCING

HEAD IN FEET	POUNDS PER SQ. INCH
0-20	14,000
20-40	13,000
40-60	12,000
60-80	11,000
80-100	10,000
100-120	9,000
120 and Over	8,000

TABLE IV

EFFECTIVE DEPTH TO STEEL

Diameters less than 6' Total Thickness	−2"
Diameters from 6' to 12' " "	−2½"
Diameters 12' or more " "	−3"

Note. Longitudinal Reinforcement:
For Covered Pipe ___ 0.25 of 1%
For Exposed Pipe ___ 0.50 of 1%
Allowable Stress in Concrete 600 lbs. per sq. in.

EXPLANATION

1. From the Coefficients in Table I determine for each Linear Foot of Pipe Total Moment M in Foot Pounds and the Thrust T_1 in Pounds due to all Loads other than the Hydrostatic Pressure.
2. Determine the Thrust T_2 due to the Head measured from the Hydraulic Gradient to the Inside Top of Pipe $T_2 = 62.5 \times Hd.$ in Ft. $\times$ Inside Radius of Pipe in Ft.
3. Determine Total Thrust T from $T = T_1 + T_2$
4. Determine the Eccentricity "e" in Inches from $e = \frac{M}{T} \times 12$.
5. Determine the Relation q/d when "d" equals Depth to Steel in Inches.
6. Determine $\frac{e}{d}$.
7. Enter Diagram 1 at Top with an assumed percentage of Steel, proceed Vertically Downward to the Value of q/d determined in 5, Read Value of k at Left.
8. Continue Vertically Downward on the percentage first assumed to the Line corresponding to the Value of k determined in 7 thence to the Right to the Intersection with the Line corresponding to the Value $\frac{e}{d}$ determined in 6, thence Vertically Upward to the Value of k determined in 7 and Read the Stress in the Concrete, Continue Vertically Upward and read the Stress in the Steel. If above or below allowable Stresses assume a new percentage and repeat.
9. When the Hydrostatic Pressure is sufficient to require Two Layers of Steel, determine the Relation q/d where d' equals the Depth to the Steel in Inches from the nearest Face of the Concrete.
10. In Diagram 2 determine the Limiting Value of q/d for the Relation of q/d determined in 9. This Value of q/d indicates the Point at which the change from Single to Double layers of Steel is made. Compute the Area of Steel required for Double layer Reinforcement by the Hydrostatic Head on Center of Pipe.
11. The required Spacing of Steel applies to the Shorter Arc of Vertical Curves in the Pipe. Diagram 3 gives the Maximum allowable percentage for different Sizes of Bars for variable effective Depths to Steel.
12. Maximum spacing of Bars = 1.5 d (Maximum 12")

Note

100 Circumferential Bars on 45° Axis of Pipe
Area of Section $3.356t^2 - \pi r^2$

FIG. 160.—Method of designing monolithic concrete siphon barrels.

resistance to external loads properly applied; easily drilled and tapped for service connections; joints can be made without specially skilled labor; flexible joints; no expansion joints are needed; and the initially high Hazen and William's coefficient of 140 does not reduce materially with age. Among its disadvantages may be included relatively low structural resistance to flexural stresses that cause breakage when the pipe is moved or undermined under pressure; the pipe is easily punctured by excavating tools; the durability of rubber in the joints is uncertain under all conditions, being attacked by gasoline and petroleum products; the material may be corroded by acids and by sulphates in the soil; and its dialectric properties prevent the thawing of frozen pipes by electricity.

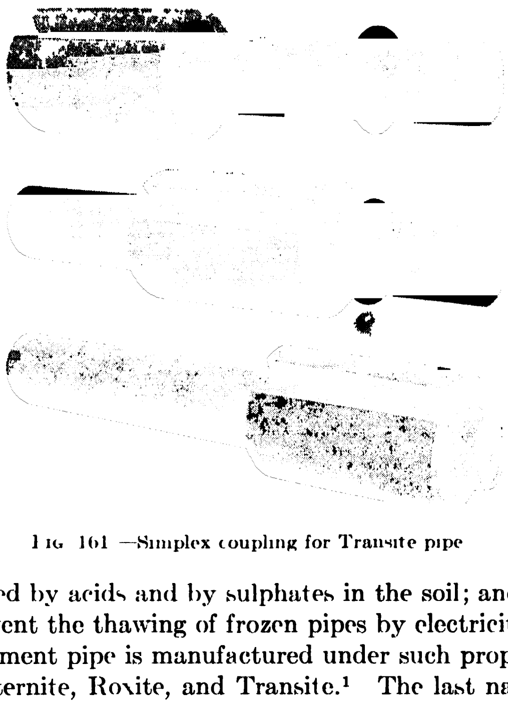

FIG. 161 —Simplex coupling for Transite pipe

Asbestos-cement pipe is manufactured under such proprietary names as Century, Eternite, Roxite, and Transite.[1] The last named is manufactured in four different grades to withstand internal bursting pressures from 50 to 200 p.s.i.; in standard internal diameters from 2 to 36 in.; and in standard lengths of 5 and 10 ft. for 2 to 3½ in. diameters and 13 ft. for 4 in. and larger diameters. All pipes are finished with plain ends, the exterior of the barrel being machined to receive the coupling illustrated in Fig. 161. No special skill is required in completing the connection, as the coupling is pushed over the rubber gasket, which rolls up on the pipe, making a tight and flexible joint which can be completed either in a dry trench or under water. The pipe can be tapped and threaded for service connections in the same manner as iron pipe, and it can be cut to any desired length, shorter than standard, and at any angle, by means of a common wood saw and a miter box. Standard cast-iron specials may be connected to the pipe with sulphur-base compounds or metallic lead

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[1] Committee Report, *Jour A W W A* , May, 1937, p 607. BLAKELEY, G. W., "Transite Pipe," *Jour. New Engl Water Works Assoc.*, September, 1937, p. 317.

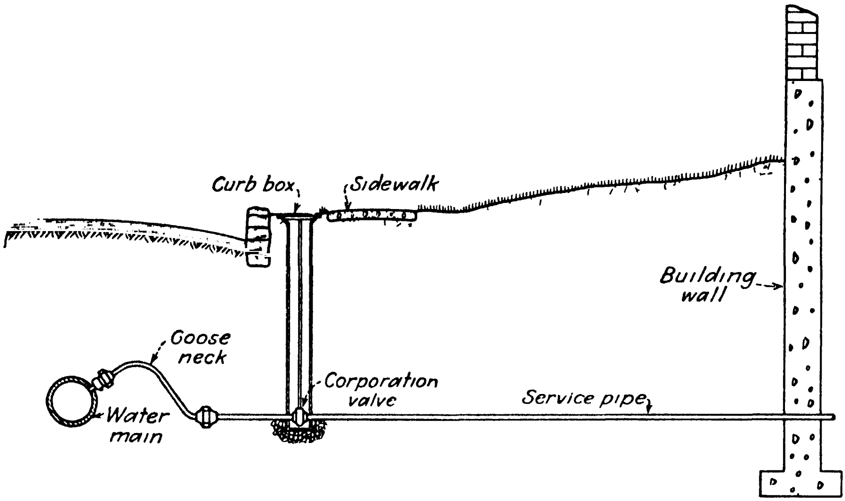

FIG. 162.—Service-pipe installation

310. Service-pipe Connections. A service pipe is usually connected to the water main in the street by a valve called a corporation cock, which should be threaded into a hole tapped into the water main. Because of movement that takes place between the water main and the service pipe, flexibility must be allowed at the junction This is obtained by the use of a "gooseneck" connection of flexible material. A typical installation of a service pipe and of a gooseneck connection is shown in Figs. 162 and 163. The service pipe leads from the water main to a valve, sometimes called a "curb stop," placed under the parking, lawn, or other place accessible from the ground surface. The service then leads to a stop-and-waste valve close to the foundation wall inside the building. If a meter is installed, it may be placed close to and on the house side of the stop-and-waste valve. Occasionally, meters are placed in vaults outside the building.

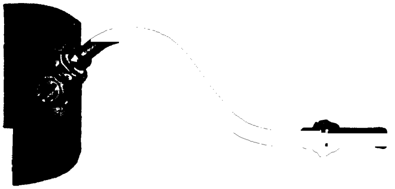

FIG. 163 – Lead gooseneck (*Courtesy of Mueller*)

Sizes of service pipes and meters should be selected to supply water at a satisfactory rate to every plumbing fixture in the building. Conditions affecting these sizes include[1] (1) pressure in the water main, (2) length of service pipe, and (3) number and types of plumbing fixtures supplied. In general, the flow into a fixture will be adequate if the pressure on it is 5 to 10 p.s.i. with the faucet wide open. Common sizes of service pipes and meters in use for residences are ⅝, ¾, and 1 in. Larger sizes are used for heavier demands. Some approximate figures for head losses in meters and services are given in Table 59.

TABLE 59.—APPROXIMATE HEAD LOSSES THROUGH METERS AND SERVICES (Saint Paul Water Department Engineering Division)[1]

Rate of flow, g.p.m.	Head losses in pounds per square inch per 100 ft. of service pipe											
	Meter size, in.						Size of pipe, in.					
	⅝	¾	1	1¼	1½	2	⅝	¾	1	1¼	1½	2
10	6	2.0	0.5	0.5	0.2		30.0	11.0	3.0	0.9	0.3	0.1
12	9	3.0	1.0	1.0			41.5	15.0	4.5	1.4	0.5	
14	11	4.0	1.5	1.5			55.5	19.5	6.0	2.0	0.7	
16	14	5.0	2.0	2.0			71.0	24.5	7.5	2.3	0.9	
20	22	8.0	3.5	3.0	1.0	0.4		37.5	11.5	3.7	1.4	0.4
24		11.0	5.0	4.0				53.5	16.5	5.0	2.2	
30		19.5	8.0	7.0	2.0	0.9			25.5	7.5	3.5	0.9
40			14.0	12.0	4.0	1.5			48.0	14.8	6.5	1.6
50			22.0		6.0	2.5				22.0	10.0	2.4
75					14.0	5.5				45.6	22.4	5.3
100					25.0	10.0					39.0	9.5

[1] RODIEN, G., "Sizing and Installation of Service Pipes," *Jour. A.W.W.A.*, May, 1946, p. 637.

311. Materials for Service Pipes.[2] The pipe leading from the distributing main of the public water supply to the plumbing system is known as the "service pipe." Materials most commonly used for service pipes include:

1. Copper, either iron pipe size, or copper tubing.
2. Lead, either unlined or lined with tin.
3. Galvanized iron, either unlined or lined with tin, lead, or Portland cement.
4. Brass.

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[1] See also BUSHWAY, W. B., "Practice of Determining Sizes of Service Pipes and Meters," *Jour. New Engl. Water Works Assoc.*, December, 1944, p. 317.

[2] See also CHASE, E. S., "Applicability of Various Service Line Materials," *Jour. A.W.W.A.*, August, 1939, p. 1323. "Service Pipes," Superintendents Question Box, *Jour. A.W.W.A.*, Vol. 18, p. 380, 1927. MUELLER, O. B., "A Comparison of Service Pipe Materials," *Jour. A.W.W.A.*, Vol. 16, p. 438, 1926.

5. **Cast iron, either unlined or lined with portland cement.**
6. **Wrought iron.**
7. **Plastic.**

Copper tubing is highly regarded because of its corrosion-resistant properties, its flexibility, the ease of its installation, and its relatively low resistance to flow. It can be attached to the main without the use of the conventional gooseneck, the flared end of the tubing being connected directly to the corporation cock without threading. The tubing can be pushed underground by first pushing a steel pipe and then pulling the copper tubing through the steel pipe. The steel pipe is withdrawn, leaving the copper tubing in place. Copper is insufficiently soluble in potable water to present a hazard to health.[1]

Lead possesses many of the advantages of copper tubing in that it is highly resistant to corrosion, is flexible, and has a high hydraulic coefficient of flow. It is generally considered to be unsuitable where high pressures may be encountered, and the subtle physiological dangers from lead poisoning have created some prejudice against its use. Lead pipes that conform to all acceptable specifications sometimes fail,[2] either because of "creep" or as a result of intercrystalline cracking. The finer the grain of the lead, the greater the tendency to creep; too coarse a grain is conducive to intercrystalline cracking. The ability to resist intercrystalline cracking and, at the same time, secure coarse grains may be improved by alloying the lead with antimony, bismuth, tin, cadmium, and certain other metals.

Galvanized iron or steel pipe, connected to the water main by a lead gooseneck, is probably the least expensive material for a service pipe. As the material is not highly resistant to corrosion, it should be used only in temporary installations or where there will be no undue exposure to corrosive conditions.

Cast iron, brass, and other metallic alloys may be used to resist corrosion or for other reasons, but all alloys will not resist all corrosive attacks; *e.g.*, brass is unsuitable in the presence of sea water. Cast iron is used only for services 2 in. in diameter or larger.

312. Plastics. Plastics are, in general, synthetic resins of high molecular weight, polymerized from simple compounds by heat, pressure, and catalysis. Plastics used in the manufacture of pipes and tubes belong principally to polyvinylidine chloride and cellulose acetate-butyrate types. The three plastics principally used for pipes and tubes are Saran, Tenite, and Plastitube.

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[1] See also HALE, F. E., "Relation of Copper and Brass Pipe to Health," *Water Works Eng.*, Vol. 95, pp. 84, 139, 187, and 240, 1942.

[2] JONES, B., "The Mechanical Properties of Lead Water Pipe," *Engineering*, Mar. 18, 1938, p. 285.

Saran[1] is a flexible yellowish translucent polyvinylidine chloride plastic. It melts at 310°F., will withstand water temperature up to 170°F. continuously, is hydraulically smooth, heat conductivity is 0.00022 times that of copper, coefficient of expansion is 0.000083, and its weight is about 125 lb. per cu. ft. Black Saran pipe is available in sizes from ½ to 4 in. It will withstand bursting pressures up to 1,500 p.s.i., depending on the diameter and thickness. Joints may be threaded as for metallic pipe, or they may be made by autogeneous welding.

Plastitube and Plastiflex are plastic substances extruded from cellulose acetate butyrate. Plastitube is rigid and comes in 12-ft. lengths. Plastiflex is flexible and comes in 60-ft. coils in sizes from ⅛ to 2 in. outside diameter. Tenite is a similar substance but is not well suited for pipe as it offers a fire hazard about equal to that of bulk newsprint.

Advantages claimed for plastics, particularly for Saran, include freedom from corrosion; freedom from damage due to freezing and thawing of water in closed pipe; acid resistant to solutions up to about 10 per cent concentration; light weight, about 11 per cent that of cast-iron pipe; easy to bend and to join; has adequate strength; unaffected by age, sunlight, or weather; resistant to shock; good electric insulator; resilient and flexible; and some may be transported in coils. Saran is not recommended for temperatures above 180°F.

The record of experience with plastics is brief.

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[1] See also DAWSON, F. M., and A. A. KALINSKE, "Studies Relating to Use of Saran for Water Pipes in Buildings and for Service Pipes," *Jour. A. W. W. A.*, August, 1943, p. 1058.

CHAPTER XIX

VALVES, GATES, HYDRANTS, AND METERS

313. Valves. Valves are used in waterworks practice on pipes to control the flow of water, to regulate pressures, to release or admit air, and for other purposes. For each purpose some type of valve is best suited. Additional types of valves are used in plumbing, in the generation and control of steam, and for other purposes. Losses of head in valves and fittings can be computed from the information in Table 60.

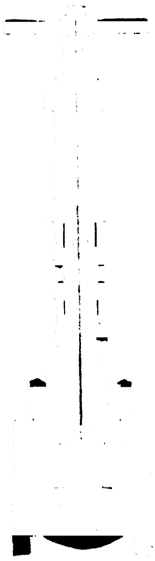

FIG. 164.- Hydraulically operated gate valve. (*Courtesy of Crane Co.*)

314. Gate Valves. Gate valves are the type of valve most frequently used in waterworks. They possess the advantage over most other types of valves, used for the same purpose, of combining relatively low cost and offering almost no resistance to the flow of water when the valve is wide open. Standard specifications for gate valves have been adopted by the American Water Works Association.[1]

Gate valves used in water-distribution systems are usually made of cast iron with brass mountings. They are usually of the solid-wedge type, illustrated in Fig. 164, or of the double-disk type, illustrated in Figs. 165 and 166. Valves of this type are available with threaded, flange, or bell-and-spigot ends. The valves shown in the figures cannot be used with complete satisfaction on sizes of pipe larger than about 16 in. in diameter when the pressures are greater than 100 p.s.i. unless a by-pass is provided, as illustrated in Fig. 167, to reduce the force necessary to open or close the valve. Gate valves of small size, buried underground, may be operated from the surface by the insertion of a "key" through a stop box with its opening at the ground surface above the valve, as illustrated in Fig. 168. Larger valves placed in underground valve chambers may be opened or closed through gearing, as illustrated in Fig. 169. Where high water pressure is always available, whether or not

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[1] Standard Specifications for Gate Valves for Ordinary Water Works Service, adopted Apr. 28, 1938, *Jour. A.W.W.A.*, Vol. 31, 502, 1939.

TABLE 60.—HEAD LOSS IN VALVES, FITTINGS, AND SERVICE PIPES

Computed from expressions

$$H = K\frac{V^2}{2g} \quad \text{or} \quad H = k\frac{Q^2}{d^5}$$

where H = head loss, ft.

V = velocity, f.p.s., in a pipe whose nominal diameter is the same as that of the valve or fitting shown in the table

Q = flow, g.p.m., in a pipe whose diameter is d in inches

K and k = constants shown in the table

Valve, fitting, or pipe	K	Source	Valve, fitting, or pipe	k (note 5)
Gate valve:			Service pipe:[6]	
Open	0.19	1	$\frac{5}{8}$-in. lead	0.00035
$\frac{3}{4}$ open	1.15	1	$\frac{3}{4}$-in. lead	0.00036
$\frac{1}{2}$ open	5.6	1	1-in. lead	0.00039
$\frac{1}{4}$ open	24	1	$\frac{3}{4}$-in. copper	0.00033
Globe valve, open	10	2	1-in. copper	0.00034
Angle valve, open	5	2	$\frac{1}{2}$-in. gal. iron[7]	0.00036
90-deg. ell. short radius	0.9	3	$\frac{3}{4}$-in. gal. iron[7]	0.00036
Medium radius	0.75	2	1-in. gal. iron[7]	0.00035
Long radius	0.6	2	Corporation cocks:	
Return bend	2.2	2	$\frac{1}{2}$-in. with tailpiece for lead	0.00127
45-deg. ell	0.42	2	$\frac{1}{2}$-in. with $\frac{3}{4}$-in. copper adapter	0.00077
$22\frac{1}{2}$-deg. ell (18 in.)	0.13	1	$\frac{3}{4}$-in. with $\frac{3}{4}$-in. copper adapter	0.0023
Tee	1.25	4	$\frac{3}{4}$-in. with tailpiece for lead	0.0020
Reducer (V at small end)	0.25	1	1-in. with 1-in. copper adapter	0.0029
Increaser = $0.25\,(V_1^2 - V_2^2)/2g$	. . .	1	Curb stops:	
Bellmouth reducer	0.10	4	$\frac{3}{4}$-in., lead service	0.002
Shear gate, open (orifice)	1.80	1	$\frac{3}{4}$-in., copper service	0.0012
Sluice gate in 12-in. wall, submerged	2.35	1	1-in., lead service	0.001
Passing branches or openings	0.03	1	1-in., copper service	0.00059
			Yokes:	
			$\frac{5}{8}$-in. rams horn (new)	0.0035
			$\frac{5}{8}$-in. rams horn (old)	0.0135
			$\frac{5}{8}$-in. straight line	0.00106
			$\frac{3}{4}$-in. rams horn	0.0029
			Stop-and-waste valves:	
			$\frac{1}{2} \times \frac{3}{4}$ in. compression valve	0.0021
			$\frac{5}{8} \times \frac{3}{4}$ in. compression valve	0.0030
			$\frac{3}{4} \times \frac{3}{4}$-in. compression valve	0.0048
			1×1 in. compression valve	0.013

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[1] *Univ. Wis. Bull.* 252.

[2] Crane Co., *Bull.* 405.

[3] *Univ. Texas, Bull.* 2712.

[4] Burns and McDonnell, *Water Works and Sewerage*, May, 1937, p. 196.

[5] Values of k are taken from *Jour. A.W.W.A.*, May, 1932, p. 631.

[6] The head losses in service pipes are per foot length of pipe.

[7] Presumably new pipe.

the valve is open or closed, the use of a hydraulic cylinder is desirable, as illustrated in Figs. 164 and 238.

In opening the valve in Fig. 165 the disk is raised by a nut which climbs the stem as the latter is turned. This is known as the "nonrising stem," the threaded portion of the stem being protected within the bonnet of the valve, as shown in the figure. It is the most common form used in valves placed underground. In the rising-stem valve, illustrated in Fig. 166, the hub of the valve wheel is threaded, and as it is turned the stem moves, carrying the disk with it. This type of valve is used where the valve is accessible for inspection and the threads can be easily lubricated. In quick-opening valves the disk is moved by means of a handle, like a pump handle, as illustrated in Fig. 170. It is suitable only for smaller sizes of pipe. Too quick opening of large valves may be undesirable because of the excessive water-hammer pressures created.

The area of opening of a gate valve, or the rate of flow through it, does not vary directly as the ratio or percentage of opening of the valve. The rate of discharge through the valve is affected by the velocity or rate of flow through it, by the length of the pipe on each side of the valve, by the loss of head through the valve, and possibly by other factors.[1] The percentage of opening of a valve is generally considered to be one hundred times the ratio of the number of turns that the valve has been opened, to the total number of turns required to open the valve fully.

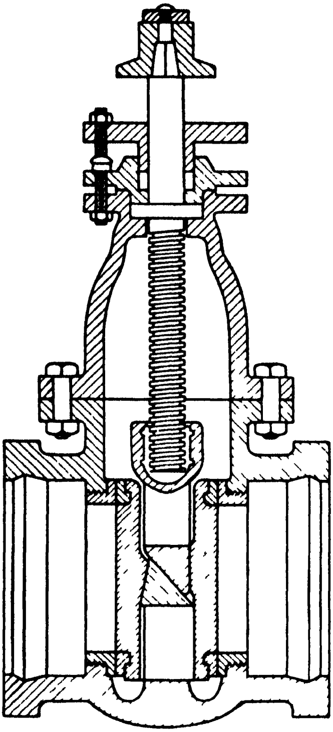

Fig. 165. Double-disk, nonrising-stem gate valve.

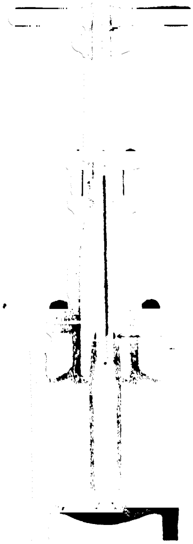

Fig. 166.—Rising-stem gate valve. (*Courtesy of Crane Co.*)

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[1] See also Angus, R. W., "The Action of Valves in Pipes," *Jour. A.W.W.A.*, November, 1938, p. 1858.

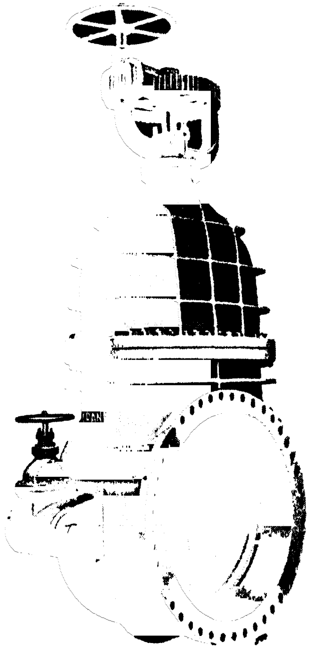

FIG. 167 Low-pressure wedge gate valve with by-pass and spur gearing. (*Courtesy of Crane Co.*)

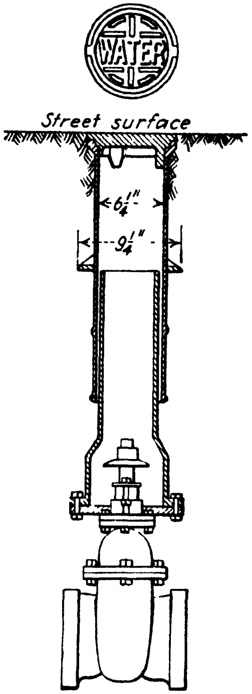

FIG. 168 Valve and valve box.

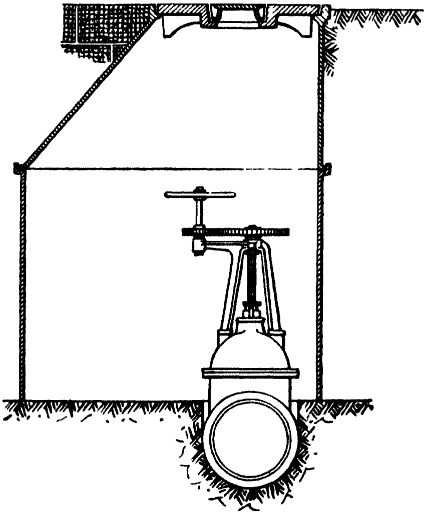

FIG. 169.—Valve, with gearing, in cast-iron manhole.

315. Globe Valves. Globe valves are used almost exclusively on pipes 4 in. in diameter, or smaller, mainly on plumbing pipes. They are rarely used on water-distribution systems. They have the advantage of being less expensive than gate valves of the same size, and they are less costly to repair. A disadvantage is the relatively high loss of head through the tortuous passage provided. A typical globe valve is illustrated in Fig 171. A relatively inexpensive type of globe valve, called an angle valve, is illustrated in Fig. 172.

316. Check Valves. Check valves are used to permit the flow of water in one direction only. The flow in the reverse direction is automatically stopped by the valve Four types of check valves are illustrated in Fig 173.

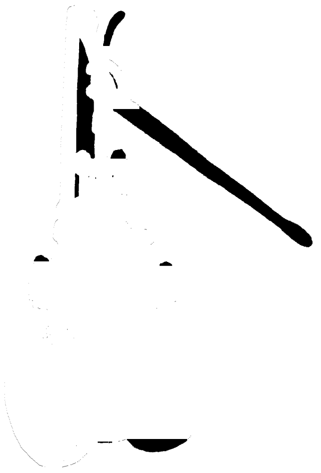
Fig. 170.—Quick-opening gate valve. (*Courtesy of Crane Co.*)

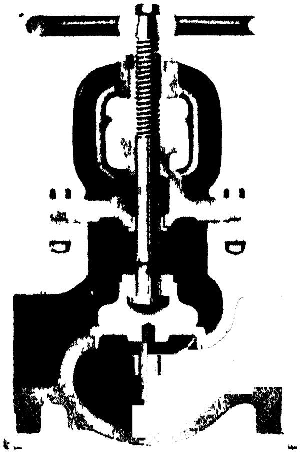
Fig 171 Section through a globe valve (*Courtesy of Kennedy Valve Mfg Co*)

Fig 172 Angle valve

317. Check-valve Slam. When a check valve closes suddenly as the result of reversal of flow of water in a pipe the force tending to stop the motion of the water is proportional to the static head on the valve.[1] This force produces an acceleration in a direction opposite to the normal direction of flow of water in the conduit. It can be shown that

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[1] See also Palmer, H K, "The Elimination of Check Valve Slam," *Water Works & Sewerage*, June, 1942, p R58

$$hAw = \frac{AwaL}{g} \qquad \text{and that} \qquad a = \frac{hg}{L}$$

where h = static head at valve at instant of reversal of flow
A = cross-sectional area of pipe
w = unit weight of water
L = length of pipe
g = acceleration due to gravity

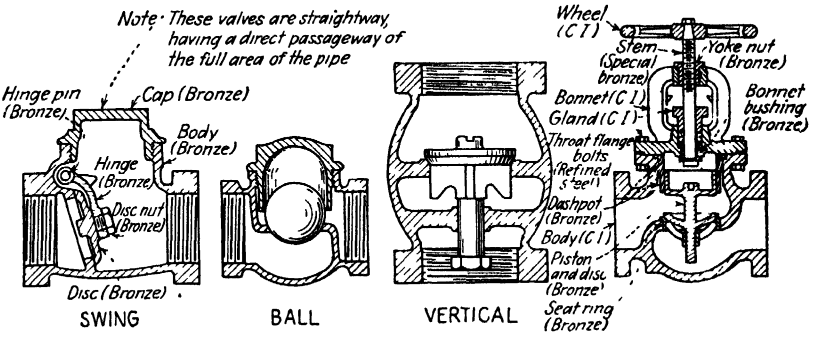


FIG. 173. Types of check valves.

Check-valve slam is minimized by many devices, some of which provide air cushions, as shown in Fig. 174, some mechanical springs, and some a slow-closing device. The intensity of pressure resulting, in part, from the sudden closing of a check valve is a function of the time of closing of the valve, the greater the time of closing, the less the valve slam.

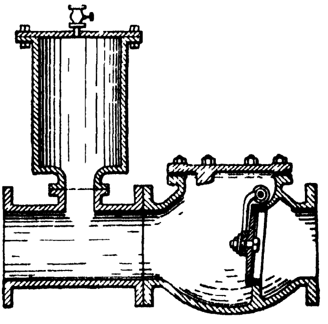


FIG. 174.—Check valve with air chamber to relieve slam.

Two types of slow-closing check valves are illustrated in Figs. 175 and 176. In Fig. 175, when water is flowing through the valve, piston B is off its seat. When the flow of water is reversed, some water flows upward through port P and presses piston B down at the same time that water pressure is transmitted through port M into the annular channel around the edge of the piston, providing a brake and cushion effect on the descending piston. Figure 176 shows a revolving-cone valve. It provides an unrestricted passage for the water when the valve is fully

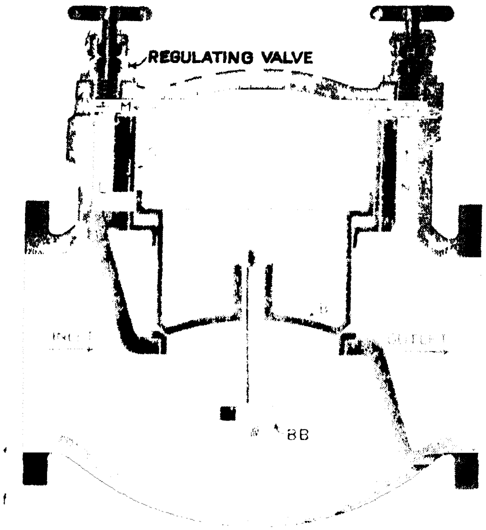


FIG. 175. Check valve designed to close slowly, without slam. (*Courtesy of Golden-Anderson Valve Specialty Company.*)

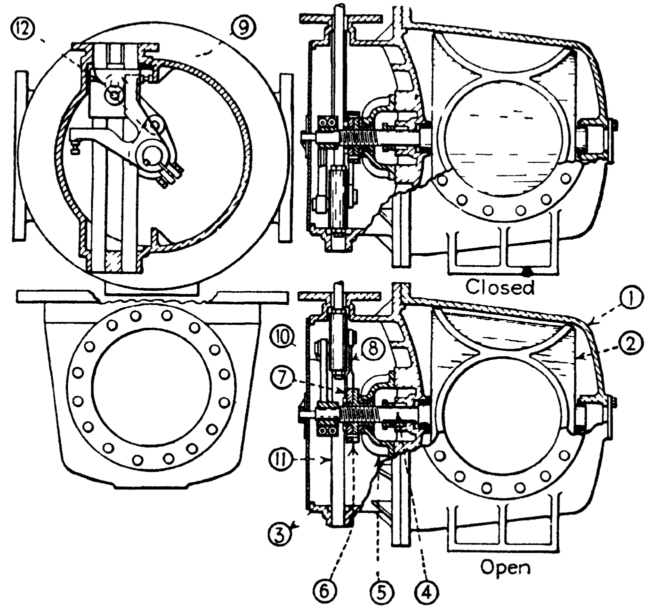


FIG. 176. Revolving-cone valve. Direction of flow in this view is either toward or away from the reader. 1, casing, 2, plug or cone, 3, cover, 4, stem, 5, yoke, 6, lifter nut, 7, lifter arm, 8, link, 9, crosshead, 10, rotator arm, 11, guides, 12, roller. (*Courtesy of Chapman Valve Mfg. Co.*)

open. It is closed by water pressure in the mechanism shown in the figure. The mechanism can be set so that the rate of closing of the valve will prevent slam.

The principle of the revolving-cone valve is used for quick-closing valves, for the control of the rate of flow, and in automatic devices.

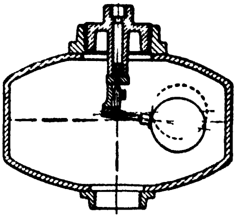


FIG 177.—Section through an air-relief valve

318. Air-relief Valves. A valve that will allow air to escape from or enter a pipe is called an air-relief valve, or sometimes if air is to be admitted only, an air-inlet valve. An air-relief valve is shown in Fig. 177. If such a valve is placed at the summit of a pipe line in which water is flowing under pressure greater than atmospheric, air collecting in the valve will allow the float to drop, thus opening a passage to the atmosphere for the escape of the confined air. If the pressure in the pipe is lowered and water escapes from the lower end, the float will drop and admit air to replace the escaping water and the creation of a vacuum will be prevented

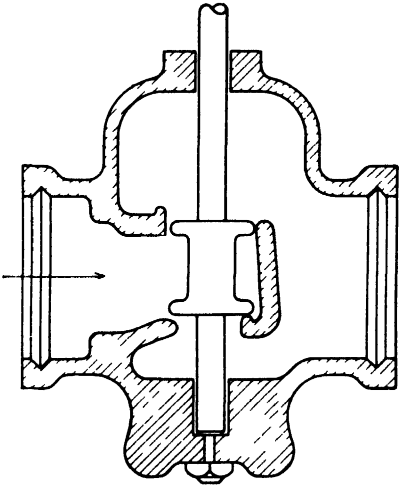


FIG. 178—Section through a balanced valve.

319. Balanced Valves. A section through a balanced valve is shown in Fig. 178. Since the upward and downward pressures against the valve disks are balanced, only friction and the weight of the parts must be moved in using the valve. The weight of the valve parts can be balanced also, so that little force is required to manipulate the valve. The principle of the balanced valve is used in many automatic devices and in other special-use valves.

320. Pressure-regulating Valves. Pressure-regulating valves are used to deliver water from a high- to a low-pressure system. Four types of pressure regulators are illustrated in Figs. 179 and 180. In Fig. 179(*A*) the desired low pressure is fixed by turning the nut at *B*, thus opening or closing the balanced valve, which is supported on the flexible disk and the spring *C*. If the pressure below the valve tends to decrease beyond the

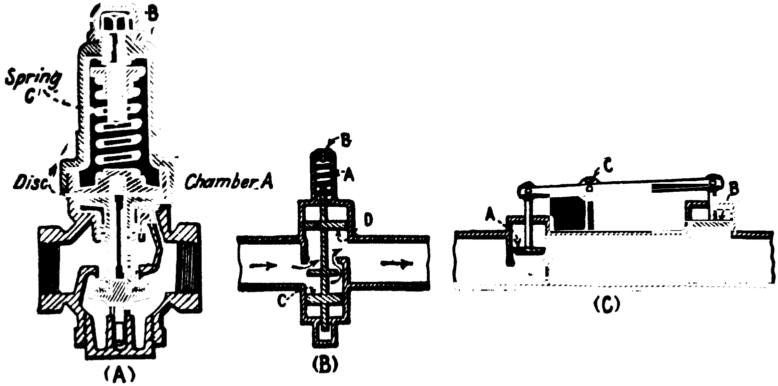


FIG. 179. Sections through pressure-regulating valves.

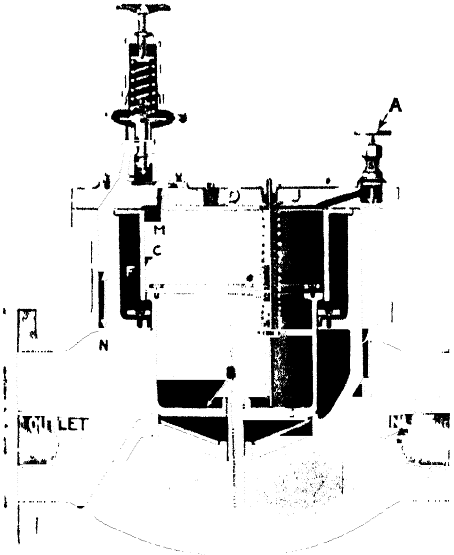


FIG. 180. Pressure-reducing valve. (*Courtesy of Golden-Anderson Valve Specialty Company.*)

predetermined amount, the pressure in the chamber at A is increased, forcing the disk up and partly closing the valve. If the pressure below the regulator becomes too low, the pressure on the flexible disk is relieved, and the spring *C* opens the balanced valve in the regulator. It is to be noted that in most pressure-regulating valves if flow stops the pressure may slowly build up on the low-pressure side of the valve.

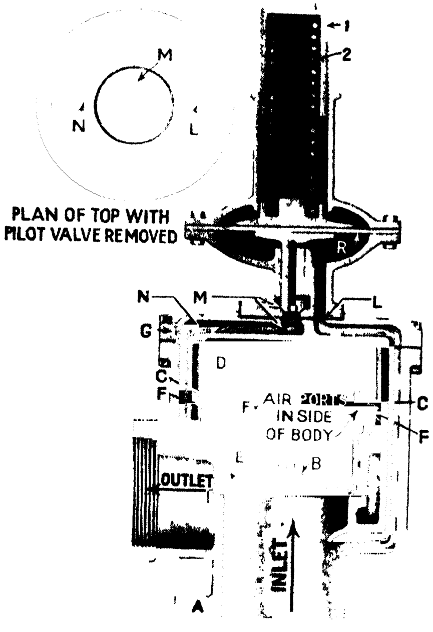
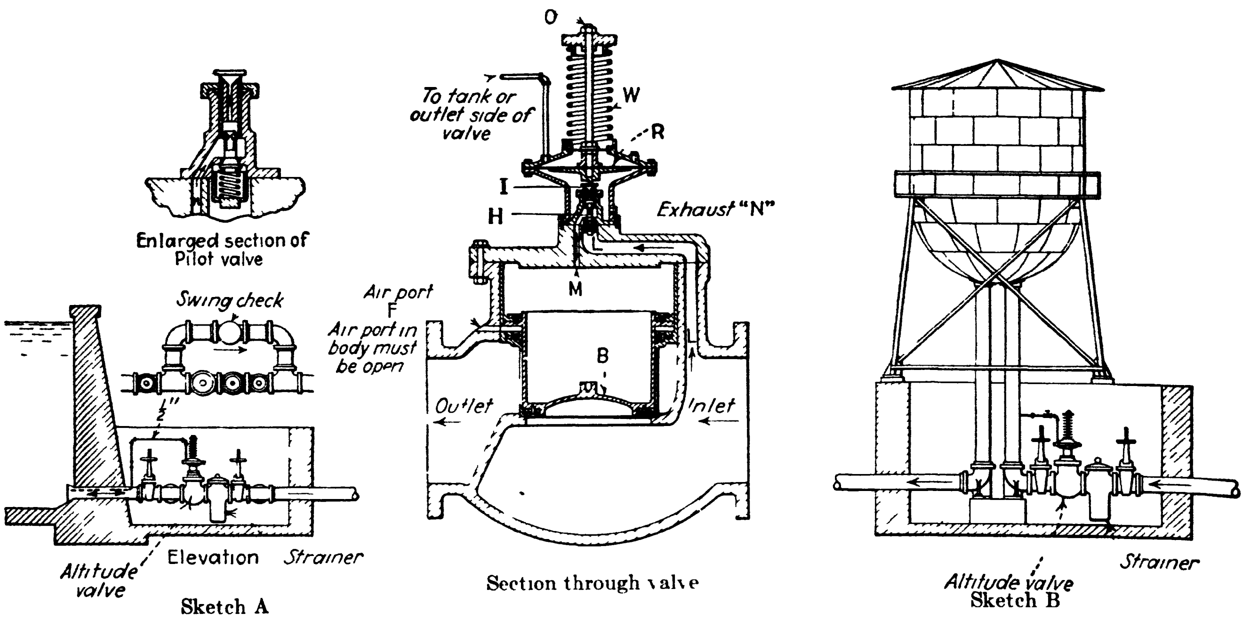


FIG. 181. Pressure-relief valve. Screwing down on sleeve 1 sets the valve for higher pressures and screwing up sets it for lower pressures. When the pressure reaches the adjustment of the valve, water passes through the port *I* and acts on diaphragm *R*, thus opening the pilot valve. This allows the water which the initial pressure has forced through the main valve *B* to exhaust through ports *M* and *N*, thus relieving the excess pressure and preventing the bursting or rupturing of lines and damage to appliances. (*Courtesy of Golden-Anderson Valve Specialty Company.*)

321. Pressure-relief Valves. One type of pressure-relief valve is shown in Fig. 181.

322. Altitude Valves. Altitude valves are used principally on the supply lines to elevated tanks or standpipes to close automatically when the tank is full and to open when the pressure on the pump side is less than that on the tank side of the valve. A section through such a valve is shown in Fig. 182. The valve shown is operated by the water pressure in the pipe line. Other valves are available that are operated by electric energy transmitted to a solenoid.



Sketch A

Section through valve

Sketch B

Fig 182.—Altitude-control valve. In the position shown in the figure the valve is closed due to the pressure above diaphragm *R* having overcome the adjustment of spring *W*. This causes the spindle head to close the pilot exhaust valve *I* and to open pilot high-pressure valve *H* thus permitting inflow of pressure above piston *B* through port *L* the pilot valve and port *M*. Since the area of the top of piston *B* is larger than the area of the bottom the larger total pressure above keeps piston *B* closed. When the static head in the tank lowers above diaphragm *R* the coil spring *W* lifts the spindle head away from exhaust valve *I* allowing it to open. This closes high-pressure valve *H* thus opening port *M* the pilot valve and port *N* to atmospheric drain. This reduces the pressure above piston *B* allowing it to open to full area. There is only a slight momentary exhaust of water from above piston *B* each time the altitude-control valve opens. The drainage is usually piped to a suitable outlet. Sketch A shows an installation in which a by-pass is used for removing storage water for distribution. Sketch B shows an installation in which a separate discharge line is used for water distribution. (*Courtesy of Golden-Anderson Valve Specialty Company.*)

323. Needle Valves. Needle valves, such as those illustrated in Figs. 78 and 183, are used for the control of water into and from large reservoirs and in the fine adjustment of flow in mechanical devices. Their principal advantage is the close control of the flow when the valve is partly open, because a large movement of the operating mechanism is required to make an appreciable change in the flow through the valve, and it is difficult to close the valve so quickly as to cause surge or water hammer.

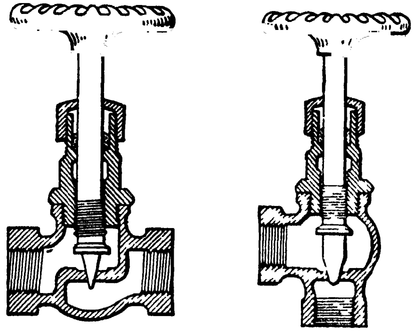


FIG. 183.—Needle valves.

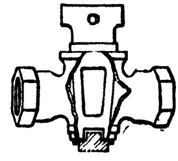


FIG. 184.—Ground-key valve.

324. Ground-key Valves. Ground-key valves, as illustrated in Fig. 184, are used only on small pipes, principally as corporation cocks on service pipes. They have the advantage of closing or opening to full flow in 90-deg. turn of the handle. Multiple-port ground-key valves, as shown in Fig. 185, are used to direct water from one channel to another without interrupting the flow of water through the valve. They are useful in filter-control devices, zeolite water softeners, etc.

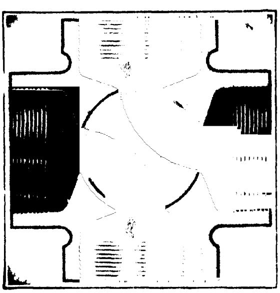


FIG. 185.—A four-way valve.

325. Mud Valves. Mud valves of the type shown in Fig. 186 are used for removing water and mud or sludge from the bottom of a reservoir.

326. Sluice Gates and Other Gates. Sluice gates are used principally at the inlet or outlet pipes entering or leaving reservoirs, the pressure on the gate frequently being against one side and in one direction only. A 60-in. sluice gate with hydraulic cylinder lift is shown in Fig. 77.

A shear gate, of the type shown in Fig. 187, is used for quickly starting or stopping the flow of water in pipes. They are not suitable for use on pipes larger than 15 to 18 in. in diameter.

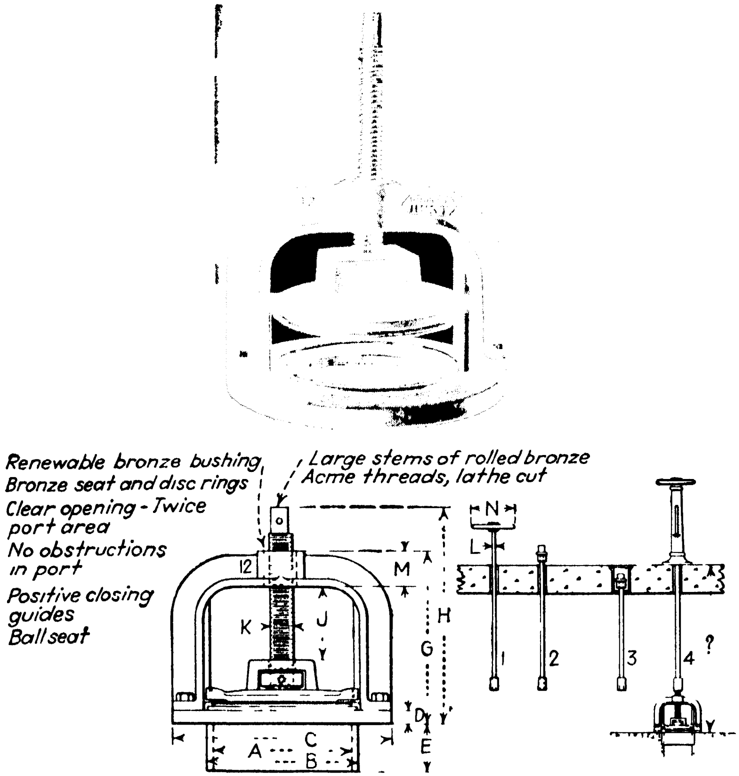


Fig. 186.—Mud valve, showing method of installation. (Courtesy of Rodney Hunt Machine Co.)

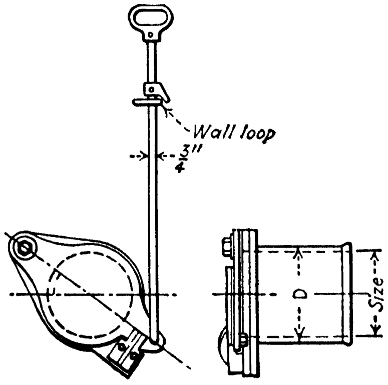


Fig. 187.- Shear gate.

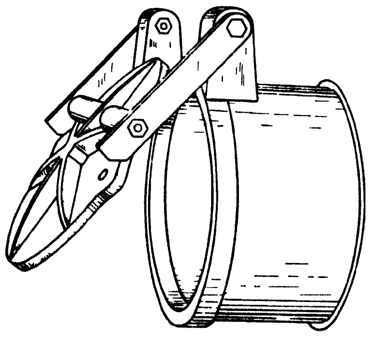


Fig. 188.— Tide gate.

A tide gate or flap valve, as illustrated in Fig. 188, is used on the outlet of a pipe to prevent backflow of water into a pipe.

327. Butterfly Valves. A butterfly valve of the type shown in Fig. 189 is used for control of the flow of water in a pipe. Such valves are usually unsuited to shutting off the flow of water completely as the moving part seldom fits closely to the seat.

328. Other Valves. Other types of valves used in waterworks include pump-control valves, surge-relief valves, and sleeve valves. Few involve principles not used in the valves described.

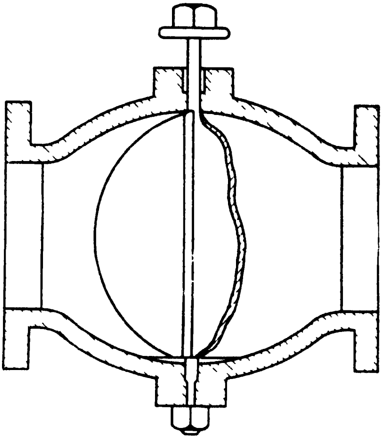


FIG. 189.- Butterfly valve.

329. Operation of Valves. Valves may be opened manually; by hydraulic, pneumatic, or other fluid pressure; or electrically. In manual operation the turning of a handwheel or lever actuates the mechanism, as shown in Fig. 167. In valves controlled by hydraulic or other fluid pressure, the valve stem, passing through a packing ring, as shown in Fig. 164, is joined to a piston in a closed cylinder. To raise the valve gate, fluid under pressure is admitted to the cylinder below the piston. At the same time any fluid in the cylinder above the piston is allowed to escape. The piston is thus lifted, and the valve is opened. When the pressure and release are reversed, the valve closes. Valves may be opened or closed by solenoids and by electric motors geared to the valve mechanism.

330. Hydrants. Standard Specifications for Fire Hydrants for Ordinary Water Works Service were adopted by the American Water Works Association on Jan. 17, 1940,[1] including standards for "Characteristics of the National Standard Fire-hose Coupling Screw Thread." Specifications for the Uniform Marking of Fire Hydrants were adopted by the American Water Works Association on June 7, 1937.[2] Various types of hydrants are illustrated in Fig. 190. They are usually made of cast iron with bronze mountings for the wearing surfaces. It is frequently desirable that a gate valve be placed on the connection to the distribution system, in addition to the main valve on the hydrant, as shown in Fig. 190c. In high-risk districts the hydrants should have a pumper connection, as shown in Fig. 191, as well as the customary hose connections.

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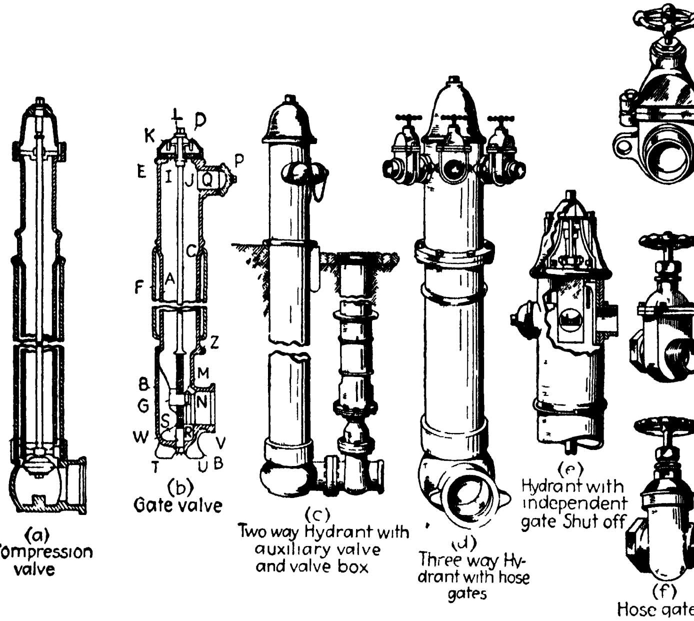
[1] See *Jour. A.W.W.A.*, August, 1940.

[2] See *Jour. A.W.W.A.*, April, 1937.

A drain for emptying the barrel of the hydrant when it is closed is essential in a cold climate to prevent the freezing of the hydrant. The drain should be connected to a storm sewer or other drainage channel or, where none is available, to a bed of crushed stone or other hard coarse material the volume of whose voids is more than three times the volume of the barrel of the hydrant.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(a) Compression valve
(b) Gate valve
(c) Two way Hydrant with auxiliary valve and valve box
(d) Three way Hydrant with hose gates
(e) Hydrant with independent gate Shut off
(f) Hose gates

FIG. 190. Types of fire hydrant The parts of hydrant (*b*) are as follows *A*, barrel; *B*, bottom; *C*, stem, *D*, follower or dome bolts, *E*, packing-plate bolts, *F*, frost case, *G*, bronze wedge nut, *H*, bronze sleeve, *I*, packing plate *J*, packing gland or follower, *K*, dome or cover, *L*, top nut, *M*, gate, *N*, bronze seat ring, *O* gate rubber *P*, nozzle cap, *Q*, bronze nozzle; *R*, bronze screwed-end drip piece, *S*, bronze drip washer, *T*, drip rubber, *U*, bronze drip nut; *V*, bronze drip cup, *W*, bronze drip bolt, *X*, bronze gate plate; *Y*, bronze gate-plate nut; *Z*, flange bolts

The flow through a hydrant should be as free and unrestricted as possible, specifications generally providing that the loss of pressure shall not exceed 1.75 p.s.i. when discharging 250 g.p.m. through each hose outlet on a two-way hydrant, 2.25 lb. on a three-way hydrant, and 3 lb. on a four-way hydrant, except when there are inside hose-gate valves, in which case the loss must not exceed 2.75 lb. for three-way and 3.5 lb. for four-way hydrants. No extra allowance is made because of the steamer connection since it and the hose connections will not be used at the same time. The discharge from hydrants has been formulated as shown on this page.

Threads for hose connections for fire hydrants have been standardized, the standard adopted by the National Board of Fire Underwriters being $7\frac{1}{2}$ threads per inch, $3\frac{1}{16}$ in. in diameter. The use of a standard thread makes possible mutual fire protection by neighboring cities; but where nonstandard threads are used, special adapters should be available which will make possible connections with any desired thread.

The National Board of Fire Underwriters requires that:

> . . . hydrants shall be able to deliver 600 g.p.m. with a loss of not more than 2.5 p.s.i. in the hydrant and a total loss of not more than 5 p.s.i. between the street main and the outlet; they shall not have less than 2.5-in. outlets and also a large suction connection where engine service is necessary. They shall be of such design that when the hydrant barrel is broken off the hydrant will remain closed. Street connections shall not be less than 6 in. in diameter and shall be gated.

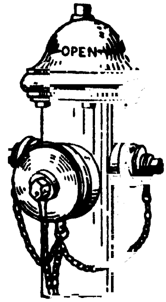

FIG. 191.—Hydrant with one steamer connection and two hose connections.

The size of a fire hydrant is designated in terms of the minimum opening of the seat ring of the main valve. It must be at least 4 in. for two $2\frac{1}{2}$-in. nozzles, at least 5 in. for three $2\frac{1}{2}$-in. nozzles, and at least 6 in. for four $2\frac{1}{2}$-in. nozzles. The rate of discharge from a hydrant can be approximated by the expression[1]

$$Q = 27d^2p^{0.5}$$

where Q = flow, g.p.m.
d = diameter of hydrant nozzle, in.
p = gage reading, p.s.i.

331. Meters. Meters used on water-distribution systems are of both the displacement and the velocity types. Displacement meters are used primarily for relatively low flows, such as those required by small and moderate consumers. A displacement meter is one that measures the quantity of flow by recording the number of times a container of known volume is filled and emptied. A velocity meter measures the velocity of flow past a cross section of known area. Types of displacement meters in use include reciprocating, rotary, oscillating, and nutating-disk meters, depending on the motion of the moving part in the measuring device. The nutating-disk meter, as illustrated in Fig. 192, is the type most commonly used on domestic services, almost to the exclusion of all other types. Turbine and Venturi meters are illustrations of velocity meters.

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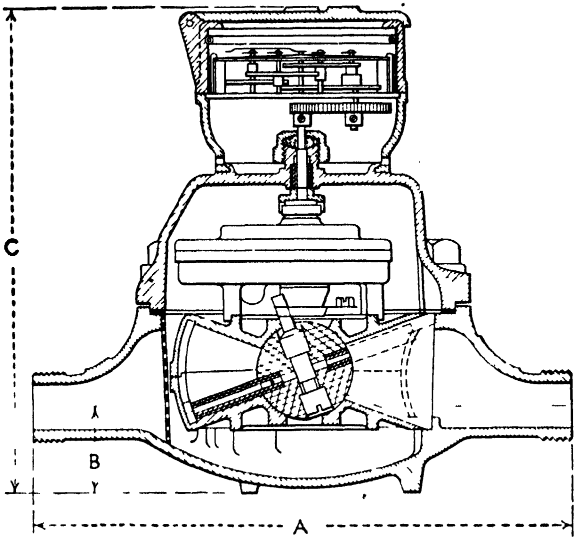
[1] See also WILSON, P S., "Hydrant Discharge Measurements," *Water & Sewage Works*, April, 1946, p R61

Nutating-disk meters may be severely damaged by passing hot water through the meter, as the disk is composed of a compound softened at high temperatures. To prevent the backflow of hot water from a plumbing system through the meter, devices are available combining a check and a safety valve or by-pass. Such devices must be of an approved type to prevent the inevitable explosion that would result from failure to relieve the higher pressures caused by the heating of the water and the generation of steam. Meters with moving parts are sometimes protected against clogging by some form of screen, usually called a fish screen. All meters should be self-cleansing to prevent the accumulation of grit and other detritus from clogging them and wearing the moving parts. Meters should be so constructed that in the event of freezing some inexpensive and easily replaceable part will break and relieve the strain on the other parts. Some meters are made partly of glass or fragile cast iron for this purpose.



Size, in.	*A*, in.	*B*, in.	*C*, in.	Width	Connections
$\frac{5}{8}$	$7\frac{1}{2}$	$1\frac{3}{16}$	$7\frac{1}{8}$	$4\frac{5}{8}$	Threaded
$\frac{5}{8}$ by $\frac{3}{4}$	$7\frac{1}{2}$	$1\frac{3}{16}$	$7\frac{1}{8}$	$4\frac{5}{8}$	Threaded
$\frac{3}{4}$	9	$1\frac{15}{16}$	$8\frac{3}{16}$	$5\frac{5}{8}$	Threaded
1	$10\frac{3}{4}$	$2\frac{13}{32}$	$8\frac{7}{8}$	$8\frac{1}{8}$	Threaded

FIG. 192. Sectional view of nutating-disk meter. ("*Tropical*" *type, Pittsburgh Equitable Meter Co.*)

Features to be considered in the selection of a meter include (1) accuracy of measurement and registration for both small and large flows, (2) capacity with reasonable head loss, (3) durability, (4) ruggedness, (5) precision of workmanship, (6) ease in repair, (7) availability of spare parts, (8) reputation of the manufacturer, and (9) cost. Tentative Standard Specifications for Cold Water Meters—Displacement Type were adopted by the New England Water Works Association on Mar. 15, 1943. These specifications were adopted as standard by the American Water Works Association on May 10, 1946.[1] The specifications cover various types and classes of cold-water displacement meters for general waterworks service and cover materials and workmanship employed in their fabrication. Some features covered by the specifications are as follows:

Head Loss: Meters 1 in. and smaller in diameter shall have head loss not exceeding 15 p.s.i., and larger meters not exceeding 20 p.s.i. when the rate of flow is that given in Table 61.

Size: Expressed in terms of nominal size of opening in inlet and outlet spuds or flanges of the meter.

Length: Over all from face to face of spuds or flanges.

Frost Protection: Devices, such as cast-iron bottom caps, shall break or yield under normal freezing conditions before meter is damaged.

Accuracy: Meters shall not register less than 98.5 per cent nor more than 101.5 per cent of water passed through the meter.

Pressure: Meters shall operate under pressures of 150 p.s.i. or less.

TABLE 61.—SIZES AND CAPACITIES OF COLD-WATER METERS—DISPLACEMENT TYPE[1]

Meter size, in.	Safe max. operating capacity, g.p.m.	Max. loss of head, p.s.i.	Min. test flow, g.p.m.	Normal test flow limits, g.p.m.	Speed of piston, revolutions, or nutations, per cu. ft.
$\frac{5}{8}$	20	15	$\frac{1}{4}$	1 20	435
$\frac{3}{4}$	30	15	$\frac{1}{2}$	2 30	250
1	50	15	$\frac{3}{4}$	3 50	115
$1\frac{1}{2}$	100	20	$1\frac{1}{2}$	5 100	50
2	160	20	2	8 160	30
3	300	20	4	16 300	15
4	500	20	7	28 500	7
6	1,000	20	12	48 1,000	3

[1] From Standards American Water Works Association.

Tentative Standard Specifications for Cold Water Meters—Current Type, covering current, velocity, and turbine meters, are published in *Journal American Water Works Association*, April, 1946. Such meters are available in sizes from 1½ to 16 in., with capacities of 100 g.p.m. up to 12,400 g.p.m., and a head loss of not more than 20 p.s.i. They are required to register within 3 per cent above or below accurate readings of the flow through the meter. Venturi meters, not covered by these standards, are in use with throat diameters greater than 48 in.

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[1] See *Jour. A.W.W.A.*, December, 1941, and April, 1946.

Tentative Standards Specifications for Cold Water Meters—Compound Type are published in *Journal American Water Works Association*, April, 1946. Such meters consist of a combination of a main meter of the current or displacement type and a small displacement meter, in conjunction with an automatic valve mechanism to direct the flow of water through the proper meter. Such meters vary from 1½ to 12 in. in diameter and will carry flows between 2 and 100 g.p.m. for the 1½-in. meter and between 32 and 3,100 g.p.m. for the 12-in. meter, with a head loss not to exceed 20 p.s.i. and a precision to register between 97 and 103 per cent, except at the region of "change over" from one meter to another, when the registration may not be less than 85 per cent of actual flow.

Tentative Standard Specifications for Cold-water Meters, Fire Service Type, are published in *Journal American Water Works Association*, February, 1947.

332. Size of Water Meters. Sizes of meters and of service pipes are closely related. Information on head losses in displacement meters is given in Sec. 331 and in Table 61. It is usually better to choose undersize rather than oversize meters because in oversize meters, although the life may be long, the accuracy is likely to be lower.

CHAPTER XX

METALLIC CORROSION

THEORIES OF CORROSION

333. Explanations. The effect of corrosion is so important in the selection of materials to be used in waterworks and in the maintenance of waterworks equipment that an understanding of its causes and of methods for its prevention should be possessed by the waterworks engineer. Hypotheses that have been advanced to explain corrosion are presented in the following sections.

334. Galvanic, or Bimetallic, Corrosion.[1] A metal when immersed in water or in the solution of an electrolyte, such as corrosive water, tends to go into solution. Cations (atoms bearing a positive charge) of the metallic element tend to migrate through the solution and plate out on the surface of an electronegative metal. It is to be noted, however, that the solution must be electrolytically neutral. Hence there must be the migration of anions (atoms bearing a negative charge) toward the anode. Camp[2] explains a typical reaction as

$$\text{Metallic element} \underset{\text{reduction}}{\overset{\text{oxidation}}{\rightleftharpoons}} \text{cations} + \text{electrons}$$

$$\underset{\text{metal}}{\text{Fe}} \rightleftharpoons \text{Fe}^{++} + 2 \text{ electrons}$$

$$\text{Fe} + 2\text{OH} \rightarrow \text{Fe(OH)}_2 + 2 \text{ electrons}$$

He lists 267 metallic corrosion reactions, with their electrolytic potentials, and gives examples of computations of quantitative computation of the corrosiveness of water on iron. Where two dissimilar metals are immersed in the same electrolyte, there is an interchange of ions between them or with the electrolytic solution. The rate of corrosion is determined, in part by the degree of electrical dissimilarity between the metals in the electrochemical series, as shown in Table 62, in part by the distance between the poles, and by other conditions. For example, higher temperatures speed corrosion reactions due to increased ionization; and

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[1] See also GLEESON, G. W., "Fundamentals of Corrosion," *Jour. A.W.W.A.*, July, 1941, p. 1249.

[2] CAMP, T. R., "Corrosiveness of Water to Metals," *Jour. New Engl. Water Works Assoc.*, Vol. 60, pp. 188 and 282, 1946.

gases released from solution, diffusion and turbulence, and lowering of viscosity, all tend to enhance the migration of iron ions. The voltages shown in the table are those which will exist only under the conditions

Table 62.—Single Electrode Potential of Certain Elements

(Showing electrode potentials at 25°C. with respect to the normal hydrogen electrode. Of two electrodes the higher one in the table forms the anode, and the lower the cathode. The electromotive force between the two is the difference between their respective potentials.)

Element	Ion	Potential	Element	Ion	Potential
Potassium	K^+	−2.9241	Mercury	Hg	
Calcium	Ca^{++}	−2.763	Hydrogen	H^+	0.000
Sodium	Na^+	−2.7146	Antimony	Sb	+0.226
Magnesium (18°)	Mg^{++}	−1.866	Bismuth	Bi^{---}	
ZINC	Zn^{++}	−0.7618	Arsenic	As	
Aluminum	Al		COPPER	Cu^{--}	+0.3441
Manganese	Mn		Copper	Cu^-	+0.522
Chromium	Cr^{++}	−0.557	Iodine	I^-	+0.5345
IRON	Fe^{++}	−0.441	Silver	Ag^-	+0.797
Cadmium	Cd^{++}	−0.4013	Mercury	Hg	+0.7986
Cobalt	Co^{++}	−0.29	Bromine	Br^-	+1.0648
Nickel	Ni^{++}	−0.231	Chlorine	Cl^-	+1.3583
TIN	Sn^{++}	−0.136	Gold	Au^{---}	+1.36
LEAD	Pb^{++}	−0.122			

Table 63.—Cathodic Overvoltage of Certain Metals in Normal Sulphuric Acid Solution[1]

Metal	H_2 in H_2SO_4 solutions	Metal	H_2 in H_2SO_4 solutions
Zinc	−0.75	Silver	−0.29
Tin	−0.49	Iron	−0.27
Lead	−0.42	Aluminum	−0.19
Gold	−0.36	Nickel	−0.14
Copper	−0.33	Platinum	−0.12

[1] Newberry, E., *Mem. Proc. Manchester Lit. Phil. Soc.* 61 (II, III), *Mem.* 9; *Chem. Abstracts* 12, p. 2496; see also Taylor, H. S., "Treatise on Physical Chemistry," Vol. 2, pp. 814–819, D. Van Nostrand Company, Inc., New York, 1924.

shown. They may change widely if the conditions in the solution are changed.

Bimetallic galvanic action is not the explanation of all corrosion, nor will one of two dissimilar metals placed in the solution of an electrolyte always corrode. The effects will depend on the nature and concentration of the electrolyte in the solution; on the nature of the metals involved, particularly if they are alloyed; on other factors, such as internal metallic strain, turbulence, temperature, and aeration; and on protective coatings formed by the metals involved. For example, the corrosion of rivet heads is an example of galvanic corrosion caused by an electric current set up between two portions of the same metal of which one part is not under stress and the other part is under stress.

335. Hydrogenation. Water is an electrolyte, dissociating into H^+ and OH^- ions. The hydrogen ions, acting similarly to metallic anions, will migrate to and plate out on a metallic surface immersed in the water. The surface may become depolarized by the combination of the hydogen atoms with dissolved oxygen to form water, by their combination with other hydrogen atoms to form hydrogen gas, or by their combination with other substances in solution. In order to maintain electrical neutrality in the solution, ions of the metal must be discharged into the solution to neutralize the OH^- ions released by the dissociation of the water. This action causes corrosion of the metal.

The film of hydrogen on the metallic surface forms a protective coating that requires additional voltage in order that metallic ions may be discharged into solution. The additional voltage required is known as "overvoltage." It is a measurable quantity, varying in accordance with the conditions in the solution. Overvoltages of some metals under a standardized condition are shown in Table 63.

336. Electrolysis. If two dissimilar metals are immersed in an electrolytic solution and a direct electric current is passed through the cell thus formed, positively charged ions from the anode will travel to the cathode, release their charge, and plate out on the cathode. The anode is corroded.

337. Chemical Reaction. If a metal is immersed in an acid, a chemical reaction occurs, releasing hydrogen gas and forming the salt of the metal.

338. Direct Oxidation. Metals in an atmosphere containing oxygen, particularly if the metals are heated, will combine directly with oxygen in the atmosphere. A common example of this is the formation of "mill scale" or magnetite, Fe_3O_4. During this type of corrosion, oxides of iron are formed that tend to spread on the metallic surface. The rust deposit on the metal tends to inhibit further corrosion from this cause.

Oxygen is important in electrolytic corrosion because of its activity in depolarizing the cathode by removing hydrogen as it is formed there. Depolarization by oxygen permits continued corrosion by bimetallic or galvanic action. The same result is effected by agitation of water in contact with the metal since depolarization is enhanced by removal of hydrogen from the cathode.

339. Biological Action. Bacteria that cause corrosion, particularly the corrosion of ferrous metals, include the sulphate-reducing bacteria, *Vibrio desulfuricans* or *Microspora desulfuricans;* iron-consuming bacteria, such as leptothrix, spirophyllum, crenothrix, and coccobacilli; and sulphur bacteria, thiobacilli. *Oscillatoria,* a blue-green alga, has been suspected[1] of causing corrosion of iron by depolarizing the metal by removal of the natural protective coating of hydrogen.

The sulphate-reducing bacteria[2] operate in the absence of oxygen (anaerobically) to react with sulphates, and with organic compounds containing sulphur, in the soil to produce hydrogen sulphide, H_2S. The bacteria do not attack the metal directly. The hydrogen sulphide combines with iron to form compounds of sulphur and iron, or it combines with water to form sulphurous or sulphuric acid which react with the metal, with the release of hydrogen, which is again used in the bacterial metabolism. Iron ions, Fe^+, are precipitated by hydrogen sulphide as ferrous sulphide and as ferrous hydroxide, removing iron molecules from the metal. A result is the lowering of the solution pressure of iron together with removal of cathodic hydrogen which, if not removed, would have retarded corrosion. The prevention of bacterial corrosion by cathodic protection[3] is explained since the depolarization of the cathode is inhibited by the electric current.

The hydrogen-ion concentration of the soil is an important factor in this action since the bacterial destruction of proteins in the soils produces ammonia, a weak base, and the bacterial destruction of carbohydrates produces weak acids. The result of such actions is to render the soil slightly basic or to increase the pH. For bacterial corrosion of this type to proceed, the pH of the surrounding soil must be between 6.2 and 7.8. Reducing actions take place most effectively under anaerobic conditions. Such conditions are commonly found in swampy or moist organic soils, not necessarily submerged in water.

Thiobacilli cause corrosion by the oxidation of sulphur obtained from the ground or from joint compounds.[4] These organisms produce sulphuric acid which attacks the metal.

Iron-consuming bacteria remove iron that is in solution in the water, depending on concentrations of soluble iron or iron compounds of 2 p.p.m., or greater, to support their activities. The consumed iron is deposited by the bacteria as ferric hydroxide, forming a sheath or tubercule, around a central core, which may contain a small percentage of sulphide of iron. The effect is known as "tuberculation." It has been suggested that the corrosion of steel pipe may result also from the anaerobic reduction of nitrates.[1]

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[1] See also MYERS, H. C., "The Role of Algae in Corrosion," *Jour. A.W.W.A.*, April, 1947, p. 322.

[2] See also STARYKK, R. L., and K. M. WRIGHT, "Anaerobic Corrosion of Iron in Soil," American Gas Association, 1945, abstracted in *Jour. A.W.W.A.*, October, 1946, p. 1210.

[3] See also LOGAN, H. H., *Nat. Bur. Standards, Circ. C450*; and *Eng. News-Record,* May 2, 1947, p. 109.

[4] See also BECKWITH, T. D., "The Bacterial Corrosion of Iron and Steel," *Jour. A.W.W.A.*, January, 1941, pp. 147 and 152.

Unfortunately, bacterial corrosion is not self-limiting as some other causes of corrosion, and various means of inhibiting bacterial growth must be employed for prevention. These include treatment of the water to remove bacterial food, chlorination, and chloramination.

340. Natural Protective Coatings and Tuberculation. Protective coatings are sometimes produced on corroding metals as a result of the reactions involved. Ferric oxide, or iron rust, is one of the most common forms of such coatings. Its removal apparently accelerates the rate of corrosion of the metal beneath it.

Tuberculation, a common source of difficulty from corrosion, is a combination of corrosion and the formation of protective coating in localized spots. Tubercules are conelike and resemble barnacles. They are usually the result of bacterial action, growing in concentric layers, with a central portion, usually black and soft, which contains sulphide of iron. The outer layers are of ferric oxide interspersed with a hard black layer of magnetic oxide. Favorable conditions for the growth of tubercules seem to include an excess of 2 p.p.m. of iron in solution and a fairly high velocity of flow to provide adequate food for bacterial activity.

In the corrosion of ferrous metals by hydrogenation a secondary protective coating of OH^- ions is formed adjacent to the metallic surface as an accompaniment to the formation of H^+ ions. The OH^- ions may combine with substances in solution to form a protective alkaline coating effective in inhibiting further corrosion. If the pH value of the solution is low, the formation of this alkaline coating is inhibited.

Conditions tending to remove protective coatings include high velocity of flow removing the coating by erosion, temperature changes, pH changes, changes in concentration of dissolved substances, and inability of metallic surface to adsorb a protective coating.

341. Cavitation. The effects of cavitation are similar to those of corrosion but are due more to erosion. The sudden and alternate making and breaking of a high vacuum and the creation and condensation of water vapor cause a bombardment of the surrounding surfaces with particles of water and water vapor moving at a high velocity. Erosion, with the appearance of corrosion, is a result.[1]

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[1] See also Caldwell, D. H., and J. B. Ackerman, "Anaerobic Corrosion of Steel Pipe Due to Nitrate," *Jour. A.W.W.A.*, January, 1946, p. 61.

342. Selective Corrosion of Alloys. In the selective corrosion of an alloy one metal dissolves out of the alloy more rapidly than the other. The so-called dezincification of brass is a common manifestation of this phenomenon. It is assumed that the brass dissolves as a whole, but the copper ion is redeposited, leaving the metal of low strength and composed of a spongy mass of copper. The phenomenon is limited to yellow, or high-zinc, brasses in contact with waters, particularly hot waters, carrying free mineral acids or acid-forming salts in the presence of oxygen.

343. Grounding of Electric Circuits to Water Pipes.[2] The grounding of electric circuits and equipment on water pipes *as a safety measure* is permissible under various standard codes[3] and, when done according to proper standards, has not caused damage to the pipes, to the quality of the water, to property, or to life. Among the restrictions to the practice are that only the secondary distribution circuits of alternating current may be grounded to water pipes. An electrical jumper should be connected around the water meter.

Among the objections that have been raised against the practice are that pipes are corroded by electrolysis, electrical shocks have been received by workmen, the water has been given a taste or a color, and fires have been started. The American Research Committee on Grounding[4] has found no proof that any of the above results have been caused by properly installed grounds to water pipes.

METHODS OF RETARDING CORROSION

344. Methods. Metallic corrosion may be retarded by (1) cathodic protection, (2) care in manufacture of the metal through control of its purity and composition, (3) application of coatings or linings, (4) control of the environment of the metal, and (5) treatment of the water. Methods for the treatment of water to retard corrosion are discussed in Secs. 658 and 659.

345. Cathodic Protection. Galvanic and electrolytic corrosion, hydrogenation, and to some extent bacterial corrosion of metals can be retarded by cathodic protection. Cathodic protection is accomplished by connecting the metal in an electric circuit to the negative pole of a d.-c. generator,[1] the positive pole being connected to anodes buried in the ground. The electric potential of the metal is thus reduced below that of the surroundings, and electric currents will, therefore, flow from the surroundings to the metal. A diagram of the electrical connections for the cathodic protection of a steel pipe is shown in Fig. 193.

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[1] See also ODEN'HAL, C., *Jour. Am. Soc. Naval Engrs.*, Vol. 50, No. 2, p. 231, 1938, abstracted in *Jour. A.W.W.A.*, November, 1940, p. 1964.

[2] See also KNUDSEN, H. A., "Cathodic Protection of the Mokelumne Aqueduct," *Jour. A.W.W.A.*, January, 1938, p. 38.

[3] "Interim Report of Investigations, American Research Committee on Grounding," *Jour. A.W.W.A.*, April, 1944, p. 383; and MEYERHERM, C. F., "Water Works Industry's Attitude on Grounding of Stray Currents," *Jour. A.W.W.A.*, December, 1945, p. 1298.

[4] *Ibid.*

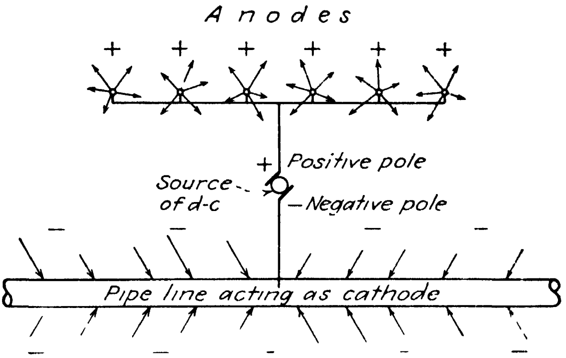

FIG. 193. Electrical circuit for cathodic protection.

The object in this method of corrosion prevention is to create an electric current that will prevent the discharge and the migration of metallic ions from the anode to the cathode. The order of magnitude of electric current for each square foot of iron surfaces[2] ranges from 1 to 15 milliamperes.[3] The voltages depend on the resistance of the circuit, but should be expected to range between 1½ and 30 volts.[4] The most desirable current density is the least that will give protection. It has been estimated that cathodic protection of water tanks is less expensive than painting the interior, and the tank does not have to be removed from service.[5]

Corrosion inhibitors, involving the addition of certain chemicals to the water, are associated with cathodic protection[1] in that the inhibitors form slightly soluble ions that affect the formation of metallic ions, thus inhibiting corrosion. Only manganese salts seem satisfactory, as salts of sodium, chromium, and zinc, whether or not the metal is also under cathodic protection, may result in the intensification of corrosion in localized areas, especially along the water line.

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[1] See also KNUDSEN, *op. cit*

[2] See also UHLIG, H. H., "Fundamental Factors of Corrosion Control," *Chem Eng. News*, Dec. 10, 1946, p. 3154.

[3] See also SCHNEIDER, W. R., "Corrosion of Steel and Its Mitigation," *Jour A.W.W.A.*, March, 1945, p. 245.

[4] See also O'BRIEN, G. L., "Tests on Cathodic Method of Protecting Steel Water Tanks," *Water Works & Sewerage*, July, 1942, p. 285.

[5] KNIGHT, G. W., "Corrosion of Steel Water Storage Tanks and Prevention by Cathodic System," *Jour. Penna. Water Works Operators' Assoc.*, 1940, p. 152.

346. Care in Manufacture. Since impurities in metal tend to create galvanic couples, resulting in corrosion, the homogenity of the metal will tend to minimize corrosion. Iron is made more homogeneous in manufacture by the process known as "spellerizing." This consists in kneading the hot iron like dough between rough rolls to produce a more uniform material. The purification of metals to remove cathodic elements of galvanic cells[2] has successfully improved corrosion resistance.

347. Alloys and Resistant Materials. Alloys of iron or steel with copper, nickel, or chromium are more resistant to corrosion than iron or steel alone. Everdur metal is a lightweight corrosion-resistant alloy consisting of copper and silicon, with other metals. Various compositions of the metal are used for tanks, gates, weirs, screens, bolts, tubing, and conduits. Special processes for the production of rustless metal are successful, and pipes and fittings of such materials are available.

For conveying acids or other highly corrosive substances, cement-lined, glass-lined (vitreous), and rubber-lined pipe; tarred, extra-heavy, cast-iron pipe; cast iron enameled inside; vitrified tile; chemically pure lead; fiber; asbestos; and other materials are used with success. High-silica pipe is resistant to acids but is more susceptible to the action of alkalies, particularly soda wastes. Pipes and fittings of high-silica iron and of the same styles and dimensions as standard cast-iron pipes are manufactured. A fiber pipe is manufactured that is resistant to both acids and alkalies, but it is structurally weak, and its use is confined to drain pipes under chemical laboratory sinks and similar locations.

348. Coatings and Linings. Coatings and linings used for the protection of iron and steel surfaces include tar or asphaltic materials, enamels, resins, lacquers, zinc, or galvanizing, metallizing, plastics, and paints. Each has shown some success in retarding corrosion.

349. Bituminous Coatings and Linings. Coal-tar enamel coatings are used extensively for the protection of steel pipe;[3] the Angus Smith coating, introduced about 1866, is used for cast-iron pipe. The latter consists of a mixture of gas tar, Burgundy pitch, oil, and resin into which the pipe is dipped while hot. About 1931, molten bituminous enamel was introduced. It is applied to the inside of the pipe while hot and the pipe is spinning around its longitudinal axis.[1] The application of enamel coatings to the exterior of pipe is accomplished by various devices that pour enamel on the pipe and spread it, with a suitable drag, overlapping spiral paths being followed by the drag or drags along the revolving pipe. Most coated steel pipe is also wrapped, frequently with asbestos felt saturated with bitumen. It may be finished with a wrapping of heavy paper.

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[1] See also EVANS, U. R., "Soluble Inhibitors," *Ind. Eng. Chem.*, Vol. 37, p. 703, 1945; and THORNHILL, R. S., "Zinc, Magnesium, and Chromium Salts as Corrosion Inhibitors," *Ind. Eng. Chem.*, Vol. 37, p. 706, 1945.

[2] See also UHLIG, *op. cit.*

[3] See Tentative Specifications for Coal-tar Enamel Protective Coatings for Steel Water Pipe, *Jour. A.W.W.A.*, January, 1940.

Asphaltic protective coatings[2] have been tested for the Asphalt Institute, with the following conclusions: (1) a wide range of performance results from the wide diversity of materials used, rather than through basic deficiencies in asphalts, (2) asphaltic coatings are less permeable to oxidizing agents than to water, and (3) asphaltic enamels of standard qualities are equal to other bituminous coatings in all respects and are superior to some.

350. Resins and Lacquers. Resins and lacquers may be classed as organic materials, in addition to bituminous materials, used for corrosion-resistant coatings and linings. Natural resins are obtained from the exudations of various plants. When applied to metals they are sometimes called lacquers. Resins may be prepared synthetically to produce thermosetting and thermoplastic coatings for the protection of metallic surfaces against corrosion. The procedure in their application involves cleaning the metallic surface and the application of the plastic resin coating by brushing, dipping, or spraying. After the evaporation of the solvents used as a vehicle, the coating is baked under controlled conditions. When lacquers and paints are used in connection with cathodic protection, they should be alkali resistant because of the alkali usually produced at the cathode.

351. Paints. Paints for the protection of metal from corrosion should be made of a metallic pigment, usually zinc or lead, mixed with an oil, usually linseed, which will dry to form a thin veneer over the surface of the metal, the pores of the veneer being filled with the metallic pigment.[1] Zinc is a particularly desirable pigment on account of its electrolytic relation to iron.

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[1] See also Bronson, D., and G. B. McComb, "Acceptance and Use of American Water Works Association, Standard Specifications for Coal-tar Enamel Protective Coatings for Steel Water Pipe," *Jour. A.W.W.A.*, May, 1943, p. 613; and Goit, L. E., "Mechanical Application of Bituminous Pipe Coatings and Linings," *Jour. A.W.W.A.*, October, 1941, p. 1723.

[2] See also Benedict, A. H., "Asphalt Protective Coating for Steel Water Pipe," *Eng. News-Record*, Feb. 26, 1942, p. 49.

Varnishes and lacquers form a japanned or enameled surface which may be allowed to dry cold or may be baked on. Such coatings are pleasing in appearance but are usually easily broken.

Aluminum and gilt paints, particularly the former, may be successful as protective coatings, but they are relatively expensive. Their appearance is sufficiently attractive to result in their frequent use for the exterior of exposed structures, such as towers and elevated tanks, the paint being sprayed, rather than brushed, on the metal.

352. Vitreous Coatings. Vitreous coatings for metal are prepared by running molten silicate into cold water. The resulting material is ground in a pugmill with clay and water. When worked to the required consistency, the metal is coated with it by dipping or brushing; when dried it is baked on to the metal.

353. Zinc Coatings and Galvanizing. Galvanizing consists in applying a zinc coating to the metal. This can be done by cleaning (pickling) the metal in acid and then dipping it in molten zinc; or the metal can be electroplated with zinc; or zinc can be applied by the sherardizing process, in which the hot iron is revolved in a drum containing zinc dust. This process produces the best results, since the threads on threaded pipe are still useful after sherardizing, which is not always the case after galvanizing by other processes.

As a result of tests by the U.S. Bureau of Standards[2] it has been concluded that over a 10-year period the rates of loss of weight of galvanized steel were from one-half to one-fifth of the rate of bare steel. For long exposure a thick zinc coating is superior to a thin one. A coating of 2.8 oz. per sq. ft. of surface prevented serious corrosion for 10 years.

354. Metallic Plating. Tin or lead plating or coating is applied in a manner similar to the hot-dip process of galvanizing, except that the iron is passed through the rolls at the same time that it is in the bath of molten tin or lead. The use of lead may be unsatisfactory in a paint or coating, as sufficient lead may be dissolved, particularly when in contact with corrosive waters, to be injurious to the health of the consumer.

Nickel and copper plating are usually applied electrolytically to the cleaned metal. Nickel, copper, and tin coatings are sometimes used in the protection of iron; but because of their relatively low electrode pressure, when the coating is once broken, galvanic action is set up between them and the iron, resulting in more rapid corrosion of the iron than if it had not been coated.

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[1] SWARD, G. G., "Paints for Water Works," *Jour. Penna. Water Works Operators' Assoc.*, Vol. 8, p. 92, 1936

[2] LOGAN and EWING, *Research Paper* 982, 1937.

Copper-coated steel pipe forms an alloy of copper and steel. It is soft, ductile, and flexible, and it can be flared, soldered, brazed, or welded. It has an ultimate strength of internal bursting pressure of 5,000 to 25,000 p.s.i., depending on the wall thickness and the diameter of pipe. Sizes $\frac{1}{8}$ to $\frac{3}{8}$ in. are available in 25-ft. coils, and $\frac{7}{16}$ to $\frac{5}{8}$ in. are available in 25-ft. lengths.

355. Metallizing.[1] Metallizing consists in spraying a coat of molten metal on any surface, metallic or otherwise, that will hold it. With careful surface preparation, proper selection of metal, and care in application, good surface protection may be obtained. The process consists essentially of passing a wire of the desired metal through a flame and driving the molten metal in a fine spray on to the prepared surface by an air blast. Any metal that can be obtained in wire form can be used. The bond is formed by minute particles of the metal penetrating into and holding in irregularities in the prepared surface which must not be smoothly polished.

Metallizing successfully protects against corrosion but, since the protective coating is only cast on the surface, it may be broken when the metal is strained in tension. The process is used also for building up surfaces worn away by friction. Metallized surfaces are particularly suited to lubricated bearing surfaces as the porous sprayed metal holds the lubricant.

356. Linings. Linings are materials that are caused to adhere to the inside of a pipe to protect it against corrosion or to make the interior smoother. The two principal materials used for linings are bituminous enamels and portland cement. Bitumastic enamels are applied as described in Sec. 349. Such linings may provide and maintain a Hazen and Williams coefficient as high as 140 to 155.

357. Cement Lining.[2] Pipes may be lined with cement in the factory, in the field, or in place.[3] A relatively simple method suitable for use in the field and requiring little equipment consists of standing a piece of pipe on end over a movable mandrel. A sufficient amount of cement, of the proper consistency, to line the pipe completely to the desired thickness is placed over the mandrel, which is then drawn through the pipe, spreading the mortar along the wall. In the centrifugal process, which is suitable for pipes up to about 48 in. in diameter, the pipe to be lined is placed in a horizontal position and revolved at a peripheral speed of about 250 f.p.m. about its longitudinal axis. The cement is spread along the inside of the pipe by means of a long trough which moves in and out of the spinning pipe. After dumping the required amount of cement into the pipe, the trough is pressed down to act as a trowel in smoothing and packing the cement, and the peripheral speed of revolution of the pipe is increased to about 600 f.p.m. The thickness of the lining is at least $\frac{1}{8}$ in. for pipes up to 12 in. in diameter and increases to $\frac{1}{4}$ in. for pipes 30 in. and larger. The centrifugal force, vibration, and rubbing of the trough produce a smooth, dense lining which adheres firmly to the inside wall of the pipe and provides a Hazen and Williams coefficient of 140 to 145.[1]

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[1] See also STEVENS, J. S., "Applications of Metallizing in Water Works Practice," *Jour. A.W.W.A.*, January, 1943, p. 53.

[2] See also American Standard Specifications for Cement Mortar Lining for Cast-iron Pipe and Fittings, A.S.A. A21.4-1939, *Jour. A.W.W.A.*, December, 1939; and "Tentative Standard A.W.W.A. Specifications for Cement-mortar Protective Coating for Steel Water Pipes 30 in. and over 7A.7," *Jour. A.W.W.A.*, January, 1940.

[3] See also HARKNESS, B., "Cleaning and Cement Lining Existing Water Mains in Place," *Water Works & Sewerage*, March, 1938, p. 182.

Large-size pipes are lined with cement by the use of molds; fittings of all sizes are lined by hand; and pipes in place are lined by drawing a mandrel through them, pushing ahead of it sufficient cement mortar to provide the necessary thickness of lining.

The mortar most commonly used consists of one part of portland cement, one part sand, and sufficient water to make a workable paste. Hydraulic cements, which include portland cement, tend to lose lime slowly when exposed to water, the lime being replaced by ferric hydroxide under proper conditions. This exchange has no detrimental effect on the cement lining and does not affect its ability to protect the pipe from corrosion.

Pipes are lined in place[2] after they have been carefully cleaned and prepared for the process. A mortar composed of one part cement and two parts sand, with sufficient water to give adequate slump for the conditions, is introduced into the pipe to be lined. A plunger is then pulled back and forth through the pipe until all the lining has been placed. Linings so placed in the field have resisted corrosion and have increased the Hazen and Williams coefficient C from about 60 to about 122 to 136.[3] Pipes up to 16 in. in diameter have been lined in place.

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[1] GIBSON, J. E., "Experience with Cement-lined Cast-iron Pipe," *Jour. A.W.W.A.*, Vol. 16, p. 427, 1926; and CARSON, H. Y., "Developments in Cast Iron Pipe," *Jour. A.W.W.A.*, Vol. 18, p. 721, 1927.

[2] See also GIBSON, J. E., "Lining Cast-iron Mains in Position with Cement Mortar," *Jour. A.W.W.A.*, May, 1940, p. 819.

[3] See also RHODES, F. A., "San Diego's Experience in Cement Lining of Cast-iron Pipe in Place," *Jour. A.W.W.A.*, January, 1944, p. 64.

CHAPTER XXI

DISTRIBUTING RESERVOIRS

358. Purposes of Reservoirs. Reservoirs are used to store water, to equalize flows, to distribute or equalize pressures, and to impound water. A reservoir on the distribution system will equalize rates of flow, will equalize pressures, and will store water for emergencies, such as fires. Such reservoirs, called distribution reservoirs, are used to adjust a variable rate of demand to a rate of supply that is not equal to the rate of demand. It is possible, by the use of such reservoirs, to reduce the size of pumps and pipes necessary to supply the district served by the reservoir, since the peak rates of demand on the pump are diminished by the reservoir. Howson[1] has shown appreciable savings in costs of distribution systems through the installation of elevated storage tanks.

In the design of reservoirs the Tentative Manual of Safe Practice in Water Distribution[2] should be consulted.

359. Classification. Reservoirs on the distribution system may be classified according to their position as surface or elevated, or according to the material of which they are built, such as steel, reinforced concrete, wood, and earth. The surface type has little or no elevation above the ground and usually consists of a tank made of reinforced concrete or a combination of reinforced concrete and earth embankments. The elevated type is built entirely above the ground surface and may be further classified into two general types as standpipes and elevated tanks.

360. Capacity.[3] The required capacity of reserve or emergency storage for purposes other than fire requirements is determined by judgment and experience after a consideration of the nature, probability, and frequency of occurrence of the hazards. Reserve storage for fire protection should be sufficient so that the required fire demand (Secs. 51 and 55) may be maintained for a period of 2 hr. in small communities up to 10 or 12 hr. in the largest communities. When the reserve storage is elevated, the amount of fire reserve in gallons may be determined by multiplying the difference between the fire demand F and the reserve fire pumping capacity P by the duration of the fire T.

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[1] Howson, L. R., "The Distribution System," *Water Works & Sewerage*, June, 1943, pp. R-79 and R-81.

[2] Published in *Jour. A.W.W.A.*, June, 1942, p. 917.

[3] See also Kennedy, R. C., "The Capacity Requirements and Design of Distribution Reservoirs," *Jour. A.W.W.A.*, November, 1940, p. 1819, and Maxwell, D. H., "Planning Capacity for Elevated Storage Tanks," *Jour. A.W.W.A.*, July, 1947, p. 644.

$$\text{Reserve storage (gal.)} = [F\ (\text{g.p.m.}) - P\ (\text{g.p.m.})]\ T\ (\text{min.})$$

In some systems it may be more economical to provide elevated storage for equalizing purposes only, fire reserve being stored in a surface reservoir. Under such a system the water in the reserve storage reservoir must be pumped into the distribution system during the emergency, such as a fire. The amount of the reserve storage is the product of the assumed fire duration and rate of fire demand for water. The reserve storage must always be kept available for immediate use, no part of the capacity provided for it being used for any other purpose.

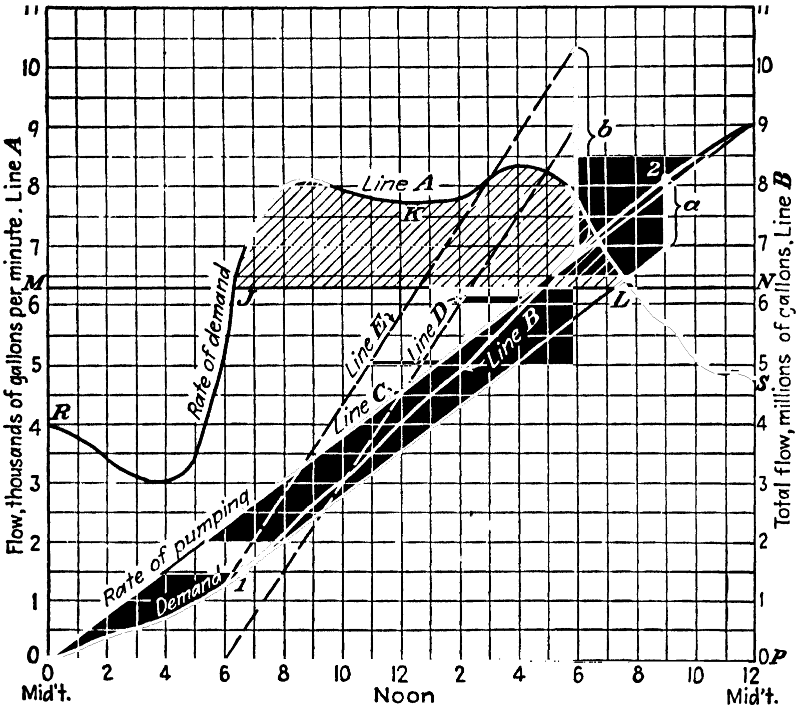


FIG. 194. Graphical determination of equalizing reservoir capacity.

361. Storage to Equalize flow. Equalizing storage is provided so that a relatively constant rate of supply can provide for a fluctuating rate of demand. The equalizing tank, or reservoir, is connected by a single pipe to the distribution system. When the rate of supply exceeds demand, water flows into the tank. When demand exceeds supply, water flows through the same pipe from the tank. The tank is said to be "floating on the line." The relation between rate of supply, rate of demand, and tank capacity is based on a study of the service required and the costs involved.

The procedure in the determination of the capacity of an equalizing reservoir floating on the line can be shown with the aid of an example. Let the hydrograph of demand of a district be shown by line *A* in Fig. 194, the abscissas of which are time and the ordinates, shown to the left, are rate of flow in thousands of gallons per min. Construct a mass diagram of the rate of demand, as shown by line *B*. Its abscissas are time and its ordinates, shown on the right, are total flow. Join the ends of line *B* by a straight line, marked *C* in the figure. This line represents the mass diagram of pumping into the reservoir, and its slope represents the rate of pumping. Draw two lines parallel to line *C* and tangent to line *B* at points 1 and 2, respectively. The vertical distance *a*, between these two lines, measured on the right-hand scale, is the required capacity of the equalizing reservoir. In the example shown it is about 1.2 million gal.

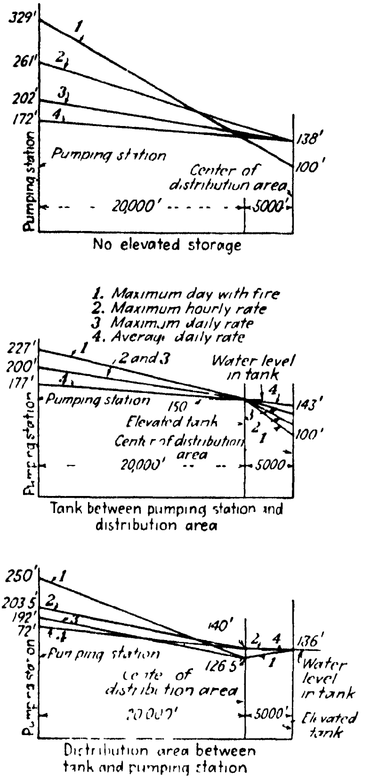
Much concerning the operation of the reservoir can be determined from a study of the figure. For example, the reservoir is

1. Full at point 1.
2. Empty at point 2.
3. Filling when slope of line *C* exceeds slope of line *B*.
4. Emptying when slope of line *B* exceeds slope of line *C*.
5. The rate of flow from or into the reservoir at any time is represented by the difference between the slopes of lines *B* and *C* at that time.

If it is desired to install an equalizing reservoir that will permit pumping at a constant rate, during the hours of greatest rate of demand, and so that the total requirements for the day will be pumped during this period, the requisite capacity of the reservoir and the rate of pumping can be determined as follows:

1. Assume the hours of pumping to be from 6 A.M. to 6 P.M.
2. Draw a straight line, such as line *D*, from 6 A.M. and zero total flow, to 6 P.M. with the total flow for the day.
3. The slope of the line just drawn represents the requisite rate of pumping. In this case it is 18 m.g.d.
4. Draw line *E*, parallel to line *D*, and intersecting line *B* at 6 A.M. The vertical distance between the intersection of line *E* with the ordinate at 6 P.M., to line *B*, represents the capacity of the reservoir on the right-hand scale. In this case it is about 3.3 million gal.

Another method of determining the requisite capacity of an equalizing reservoir for the hydrograph of demand shown in Fig. 194 is to draw a straight horizontal line representing the average rate of pumping necessary to supply the demand indicated by the hydrograph. This is line *MN* in Fig. 194. Area *OMNP* is equal to the area under the hydrograph, *ORSP*. The shaded area above line *MN* and below the hydrograph, area *JKL*, converted to units of volume, represents the required capacity of the equalizing reservoir. In this case it is 1.2 million gal.



No elevated storage

Tank between pumping station and distribution area

Distribution area between tank and pumping station

FIG. 195. Effect of elevated storage on distribution pressures.

Pumping at various rates in order to follow the demand curve must be studied for economy. Unless the demand curve is followed exactly, storage is required on the distribution system. Alternative operating schedules to be considered may include the following:

1. Installation of the storage and pumping capacity required with the pumps operating at a constant rate on the maximum day and at a reduced rate the remainder of the time.

2. Installation of the storage capacity required for an average day, in which case the installed pumping capacity must be greater than for combination 1, since on the maximum day the pumps would have to be operated at a reduced rate or rates for a part of the time on account of the reduced storage capacity and at an accelerated rate or rates for the remainder of the maximum day to maintain the demand.

3. Any other combinations of storage and pumping capacities that will satisfy the demand curve.

362. Location. Distributing reservoirs should be located centrally in, or at least as close as possible to, the district that they serve. They should also have sufficient elevation to maintain adequate pressure. The reason for a central location is to reduce friction losses in the distributing pipes by reducing the distance traveled by the water. The advantages afforded by natural elevations should be investigated.

The location of a distributing reservoir has a marked effect on the fluctuations in pressure in portions of a distribution system. This is indicated by the hydraulic grade lines shown in Fig. 195.[1] Care should be taken to locate the reservoir to obtain the most constant pressure.

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[1] See also "Elevated Storage," *Water Works Eng.*, Oct. 23, and Nov. 6 and 20, 1940, pp. 1361, 1421, and 1473.

Two or more reservoirs at different elevations may sometimes be connected to the same distribution system. The design and operation of the system under such conditions require special care.

363. Types. The walls of surface reservoirs are built of earth embankments, lined or unlined, and of masonry or reinforced concrete. The reservoirs may be open or covered. Open-earth reservoirs are the least expensive for large capacities, but they have the disadvantage of being subject to leakage. The selection of type, therefore, is usually dependent upon available funds, the value of the water stored, and subsoil conditions.

A surface reservoir, formed by building a dam across a valley, may be classed as a distributing reservoir—if its purpose is primarily to care for fluctuations in pressure and demand rather than to act as a catchment basin in which to store the water precipitated on the drainage area. The Eden Park Reservoir, forming a part of the distribution system of Cincinnati, Ohio, is an example.

The wall or walls of such a reservoir usually are dams, a discussion of which may be found in Chap. VII.

364. Depth. The depth of a reservoir necessary to give the desired capacity is influenced by the following considerations:

1. Temperature as it may affect the potability of the water, the growth of organisms, or the formation of ice.
2. The cost of land area.
3. The cost of walls.
4. The character of material to be excavated.

Shallow water is more subject to temperature effects than deep water. The effect, on shallow water, of high summer temperatures may be the imparting of taste or odor to the water, due to the encouragement of objectionable growths. Low winter temperatures may induce a relatively high reduction of capacity, due to the formation of ice. On the other hand, rock excavation may be required in order to secure the necessary depth. The cost of embankments and of masonry walls increases rapidly with increasing heights. This disadvantage may be offset by a high cost per square foot of land. For reservoirs having a capacity less than 5,000,000 gal., depths ranging from 12 to 18 ft. will ordinarily be economical.

365. Foundations. In reservoirs founded on clay, or other yielding material, the distribution of pressure over the foundation must be made uniform. This may be done by spreading the wall and column footings, as shown in Figs. 196 and 197. Rock, well-driven piles, and confined sand make satisfactory foundations requiring no special precautions in design or construction. Chemical soil solidification is reported[1] to have been used successfully in the preparation of foundations.

EARTH RESERVOIRS

366. Walls. Except in locations where the topography of the site is particularly advantageous, the walls of earthen reservoirs are formed by

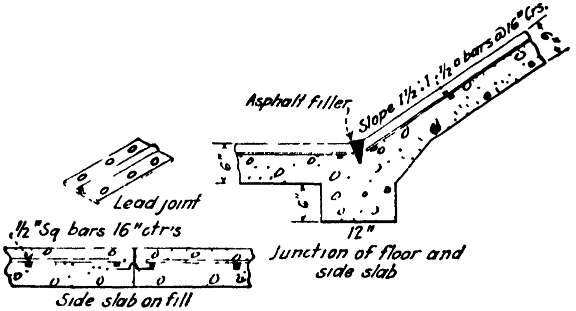
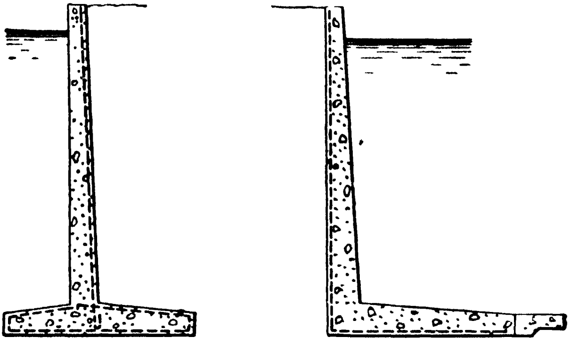


FIG. 196. Lining details, Stanford Heights Reservoir, San Francisco, Calif. (*From Jour. A W.W.A., Vol 15, p. 119.*)



T-Wall L-Wall

FIG. 197.—Types of cantilever retaining walls.

excavating to the required depth below the ground surface and using the excavated material for the building of embankments to the necessary height above the ground surface. The top width of the embankment should be at least one-fourth of the maximum total height, with a minimum width of 4 ft. Greater widths are ordinarily used. The angle of repose of the material (when saturated) will govern the maximum steepness of side slopes. Slopes steeper than 1½ horizontal to 1 vertical should not ordinarily be used. The embankments of earthen reservoirs are essentially earth dams, and the same general considerations apply to their design and construction. Because, however, of the usual proximity of distributing reservoirs to populous districts and on account of the high value of water at that location, special care should be exercised in design and construction, in order to avoid loss of water and to assure the safety of life and property.

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[1] See RIEDEL, C. M., "Chemical Soil Solidification and Chemical Sealing of Leaking Concrete," *Jour. A.W.W.A.*, September, 1945, p. 849.

Suitable materials properly placed and compacted will result in an impermeable embankment, without resort to other means. Sand or gravel, containing sufficient clay to fill the voids, is an ideal material for embankments. Thorough compacting of the material is best obtained by sprinkling and rolling. Compacting to a lesser degree is secured by the use of trucks or of teams and wheeled scrapers or of slips for placing the materials.

An effective joint between the natural ground surface and the embankment may be secured by the removal of all perishable matter, sand etc., from the original surface and below it to depths to which roots or permeable materials may extend. If rock is encountered in the foundation, a concrete cutoff wall is necessary. The wall should extend 12 to 24 in. into the rock and at least 24 in. into the fill.

367. Core Walls. In the absence of suitable materials it becomes necessary to resort to core walls to secure impermeability. Core walls used for reservoir embankment are essentially the same as those used in earth dams (Sec. 149).

368. Clay Blankets. When the substrata under a reservoir are pervious, it may be necessary to line the floor and sides with a layer of impervious material. A layer of clay, usually called a blanket, 6 in. to 2 ft. thick, sprinkled and rolled, may be used. In order to avoid the difficulty of rolling on the side slopes, the blanket may be made to connect with the core wall, if one is used.

369. Protection of Outlet Pipes. Cutoff collars or wide flanges of ample dimensions will serve to minimize the seepage along the outside surface of outlet pipes or conduits. The dimensions and spacing of the collars depend upon the amount of head on the percolating water and upon the material. The pipe or conduit must be sufficiently strong to carry the weight of the filling upon it with safety. Danger of the failure of outlet pipes, due to settlement, is minimized by supporting them on undisturbed earth.

370. Linings. Concrete is the principal material used for reservoir linings. Brick, asphalt, oiled-earth fill lined with concrete,[1] and rubble masonry are also used. To assure watertightness, bentonite clay has been incorporated with the embankment and to stop leaks, it has been applied after the reservoir has been in use.[1] The concrete of the floor is laid in blocks about 6 ft. square. The sides may be cast in blocks or in strips about 6 ft. wide, extending the entire height of the side slopes. The joints between the blocks or strips may be filled with a bituminous preparation or wooden strips, in order to increase watertightness. Tar paper is also used to separate the blocks. Construction joints are ordinarily used to reduce cracking due to shrinkage, temperature changes, or unequal settlement. The thickness of the floor in inches should be about four-tenths of the depth in feet, with a minimum thickness of 4 in. A curb should be provided at the base of the slope lining, to protect it from sliding. Stone blocks or riprap may be used as lining, to protect embankments against sloughing or against washing due to wave action.

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[1] See also BLACKBURN, D. A., "Construction of Earth Reservoir Embankments with Road Oil Linings," *Jour. A.W.W.A.*, May, 1941, p. 876.

The progress of water that has leaked through a lining may be stopped by the embankment, in which case hydrostatic pressure will be developed. If the reservoir is dewatered more rapidly than this water can escape from the embankment, pressure or uplift is exerted against the reservoir lining. Weep holes are sometimes used to prevent such a condition. Their use, however, destroys the effectiveness of the lining to gain impermeability.

Better results are obtained from linings that are reinforced than from plain concrete linings. The reinforcement prevents cracking due to the weight of the water and effects a better distribution of shrinkage cracks. An area of steel of about one-fourth of 1 per cent of the cross-sectional area of the concrete is usually sufficient. Either bars or wire mesh may be used for reinforcement. Figure 196 shows the details of the lining for the Stanford Heights Reservoir at San Francisco, Calif.

MASONRY RESERVOIRS[2]

371. Walls. The masonry walls of reservoirs are usually made of concrete and are of the gravity, cantilever, counterfort, vertical beam, or cylindrical type. The force that may be exerted against a reservoir wall is produced either by water or earth pressure.[3] If no earth fill is placed against the outside, the wall is designed to resist the water pressure when the reservoir is full. With an earth fill on the outside, the wall is usually designed to resist the earth pressure when the reservoir is empty. One of the purposes of the fill, if used, may be to effect an economy in design. In such a case, its function is to counteract the pressure of water by its passive resistance. The lateral pressure of earth is roughly about one-half the pressure of a column of water of the same height. The structural members of a wall designed to resist earth pressures are therefore proportionately smaller than those required for water pressure of a corresponding height. Such an advantage is, however, nullified if the fill is not, or cannot be, properly drained. When there is a possibility of water collecting at the back of the wall, as a result of leakage or a high groundwater table, the wall must be designed for the water pressure that may be developed on the outside. When arch or groined-arch roofs are used, it may be required to design the walls to take all or part of the arch thrust.

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[1] See also "The Action of Bentonite Clay in Stopping Leaks in Water Supply Reservoirs," *Water and Sewage*, Vol. 81, p. 14, February, 1943.

[2] See also MANNING, G. P., "Basic Principles of the Design and Construction of Reservoirs and Tanks in Reinforced Concrete," *Water and Water Eng.*, December, 1945, p. 639.

[3] See also COLE, I. G. M., "The Design of Reinforced Reservoirs and Tanks," *Water and Water Eng.*, May, 1939, p. 239.

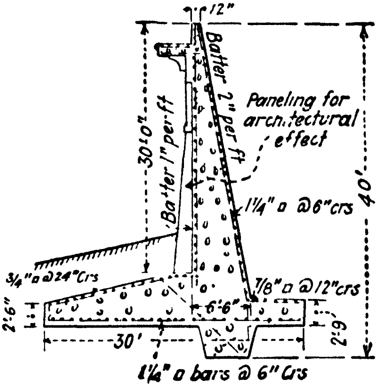

FIG. 198 Wall sections, Eastern Hills Reservoir, Cincinnati, Ohio. (*From Eng News-Record, Vol 94, p. 591, 1925*)

Gravity walls are proportioned so that they resist by their weight the overturning and sliding effects of the pressure against them. They are no longer used to any great extent in reservoir construction.

Cantilever walls consist of a base and a stem built in the shape of a *T* or an *L*, as shown in Fig. 197. The base is constructed so that its own weight plus that of the stem and a part of the retained fill develops a fixed end for the cantilever, or stem. The entire structure is proportioned so that the resultant of the vertical and horizontal forces passes within the middle third of the base width. Sliding is resisted by an adequate width of base or by a cutoff wall under the base. The thickness of the stem and the base on the side of the fill will be determined by the bending moment at the junction of base and stem, produced by the lateral thrust.[1] The maximum thickness of the toe of the base is determined by the bending moment produced by the foundation pressures on the toe, between the edge of the base and the front face of the stem. Reinforcement to resist shearing stresses is provided, if needed. The thickness of the stem, in inches, required for earth pressures will ordinarily be about equal to the height of the fill, in feet, with a minimum of 8 in. The cantilever type of wall is usually not economical for retaining an earth fill more than 20 ft.

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[1] See also YOUNG, DANA, "Bending Moments in the Walls of Rectangular Tanks," *Trans. A.S.C.E.*, Vol. 108, p. 217, 1943; and STANLEY, C. M., JR., "Concrete Reservoirs of the Vertical-beam Type," *Trans. A.S.C.E.*, Vol. 109, p. 567, 1944.

in height. Figure 198 shows a detail of a cantilever wall used in the Eastern Hills Reservoir at Cincinnati, Ohio.

A counterforted wall is essentially a contiuous horizontal or inclined slab, supported by vertical counterforts spaced 7 to 10 ft. apart, depending on the height. The counterforts should have a thickness of about one-twentieth of the height of the wall when earth pressures are resisted. The thickness of the face slab and base slab are determined by the forces to be resisted and by the span of the counterforts. The design of counterforted walls is somewhat more complicated than that of cantilever walls, but the counterforted type will show a saving in material for heights of earth of more than 20 ft. Figure 199 shows a detail of a counterforted wall used in a reservoir at Saint Paul, Minn.

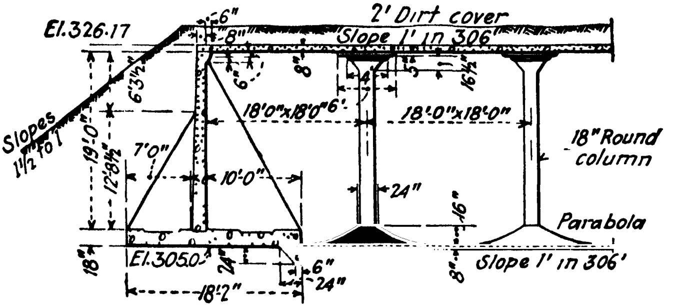

FIG. 199 High-service Reservoir at St. Paul, Minn. Capacity, 18,000,000 gal

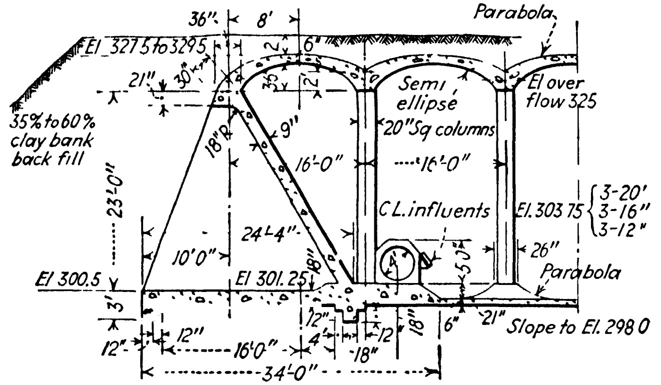

FIG. 200.—Dale Street Reservoir, St. Paul, Minn. Capacity, 30,000,000 gal.

A detail of an inclined slab-buttressed wall, also at Saint Paul, Minn., is shown in Fig. 200. This wall is designed to carry either external or internal water pressure, and no dependence is placed upon the passive resistance of the earth embankment. The arch thrust of the roof is transmitted to the buttresses through the reinforced beam at the top.

The cutoff wall under the base slab develops additional resistance to sliding.

In beam-type walls, unit lengths of the wall are designed as vertical beams supported by floor and roof. The ordinary methods of reinforced-concrete design are applicable. Keyways at the elevation of roof and floor provide simple supports for the beam. This type results in an economical design, when conditions permit its use.

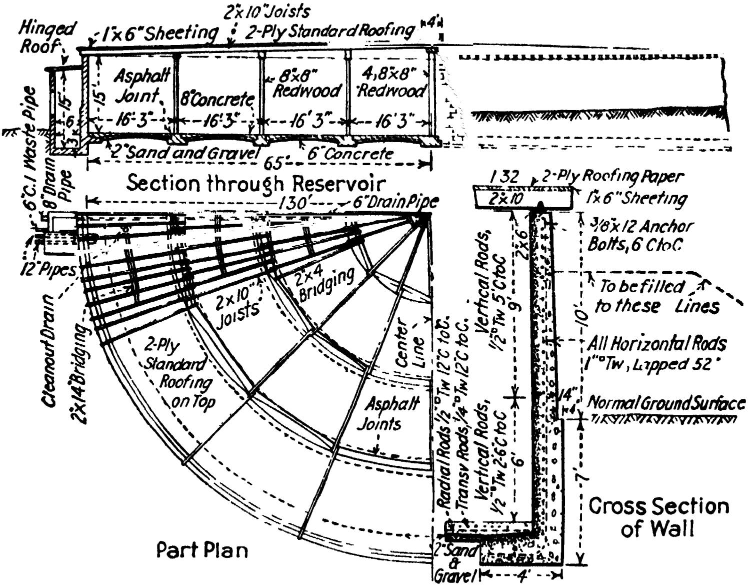

FIG. 201. Circular Reservoir, Santa Monica, Calif. (*From Eng. News-Record, Vol. 79, p.* 1107.)

When the reservoir takes a circular or ring shape, advantage may be taken of the cylinder theory of design.[1] The circular shape eliminates stress due to bending moment. The stresses carried by the wall are either tension or compression, depending on the loading. This type of reservoir will usually require the least thickness of walls, provided the diameter is not excessive. It is usually the most economical type for small capacities. Its disadvantages lie in the higher cost of form work and less economical use of land. Where reinforcing is used, the gunite process requires no outside forms in placing the concrete. Figure 201 shows the details of a circular reservoir at Los Angeles.[2] This structure is built with prestressed reinforced-concrete walls and dome roof. The stressing of the reinforcement before filling the reservoir maintains compression in the concrete when the reservoir is full. Cantilever or buttressed walls may have a circular shape in plan view. Partition walls are provided in reservoirs either to serve as baffles or to permit a part of the reservoir to remain in service while an adjacent part is being cleaned or repaired. It is important that partition walls be designed to resist water pressure from either side in order that either chamber may be emptied.[1]

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[1] See also SALTLR, G. S., "Design of Circular Concrete Tanks," *Trans. A.S.C.E.*, Vol. 105, p. 504, 1940.

[2] See also GLACE, I. M., "Pre-stressed Gunite," *Water Works & Sewerage*, March, 1945, p. 83.

372. Floors. The floors of masonry reservoirs are of three general types: (1) the square block as described under earth reservoirs, (2) reinforced slabs, and (3) groined arches. The first are used when no complications arise to foundation or uplift pressures. Otherwise, reinforced slabs or groined arches are necessary. The slabs or groined arches serve to transmit hydrostatic pressures to the columns. Watertightness is obtained, as in earth reservoirs, by wooden strips, usually of cypress, or by bituminous compounds placed in the joints of the masonry.

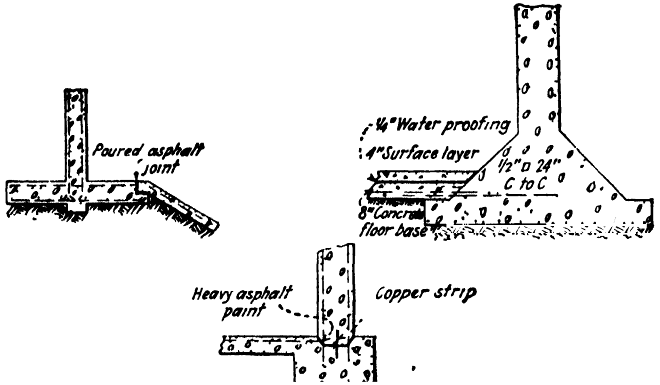

FIG. 202 Examples of connections of floors and walls.

The connection between the floor and the walls and between the floor and the columns is an important detail. A construction joint is usually necessary to provide against cracks resulting from unequal settlement. Figure 202 shows details of floor-and-wall connections.

373. Roofs. Reservoir roofs are built of wood and of reinforced concrete. The latter may be of the beam-and-girder, the flat-slab, the arch, or the groined-arch type. The usual methods may be applied in the design of wood roofs or concrete roofs of the beam-and-girder or flat-slab type. A method for the design of groined arches may be found in

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[1] A description of the failure of a partition wall in a reservoir at Baltimore is given in *Eng. News-Record*, Vol. 102, 1929.

Transactions of the American Society of Civil Engineers, Vol. 86, page 180, 1923. A section through a reservoir at Springfield, Vt., with unreinforced concrete roof is shown in Fig. 203.[1]

Wooden roofs are the least expensive but have the disadvantage of being short-lived and of high maintenance cost. Ordinary rules of timber design are used to determine the size and spacing of columns, girders, and joists. Shingles form a good covering material. The slopes of reservoir roofs are usually too flat for the successful use of ordinary prepared roofing.

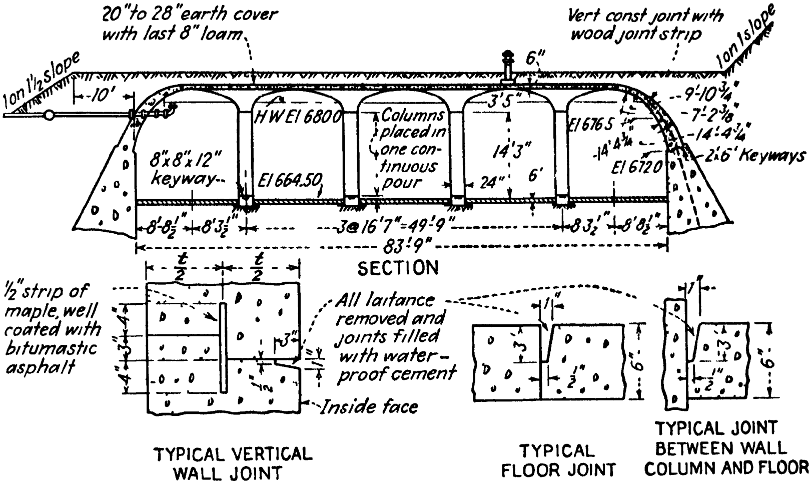
20" to 28" earth cover with last 8" loam
Vert const joint with wood joint strip
1 on 1½ slope
1 on 1 slope
6"
10'
HW El 6800
Columns placed in one continuous pour
3'5"
14'3"
El 676.5
14'4¾"
El 672.0
9'-10¾"
7'-2⅜"
14'-4¾"
2×6' Keyways
8"×8"×12" keyway
El 664.50
24"
6'
8'-8½"
8'3½"
3@16'7"=49'-9"
8'3½"
8'8½"
83'-9"
SECTION
t/2
t/2
½" strip of maple, well coated with bitumastic asphalt
4"
3"
4"
3"
All laitance removed and joints filled with water-proof cement
Inside face
1"
1"
1"
½"
6"
1"
2"
6"
TYPICAL VERTICAL WALL JOINT
TYPICAL FLOOR JOINT
TYPICAL JOINT BETWEEN WALL COLUMN AND FLOOR

FIG. 203.—Unreinforced concrete reservoir roof at Springfield, Vt. (*From Eng. News-Record, Sept.* 23, 1943, *p* 105.)

Reinforced-concrete roofs are usually designed to support 2 to 3 ft. of earth fill. The purpose of the fill is to protect the water against extremes of heat and cold or to provide for landscaping. The earth load plus 100 lb. per sq. ft. is the loading usually used in design. Figure 199 shows an example of a flat-slab roof. Figure 200 shows a detail of a reservoir having a groined-arch floor and roof. Detailed methods for the design of concrete reservoirs may be found in standard textbooks and handbooks that deal with the subject of reinforced concrete.

ELEVATED RESERVOIRS

374. Types. Elevated reservoirs are used where a tank on the ground will not supply needed pressure. The height of the elevation is fixed by the required pressure. Among the advantages of the use of elevated storage may be included the reduction of pump and of pumping cost, the reduction of the maximum pressures required at the pump discharge, and the reduction of peak demands on the pumps. There are two types of elevated reservoirs: standpipes and elevated tanks. A standpipe usually consists of a cylindrical shell, built of steel or reinforced concrete, having a flat bottom and resting on a foundation on the ground. An elevated tank is a reservoir supported on and anchored to an elevated structure or tower. It is desirable to make the horizontal cross-sectional area of an elevated tank or standpipe relatively large in order to obtain maximum storage volume with minimum pressure variations in the system resulting from the filling and emptying of the tank. There is an economical, useful depth of tank that will give the least total capitalized cost of tank and of pumping. This depth, for steel tanks, is usually about 20 to 25 ft.

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[1] From *Eng. News-Record*, Sept. 23, 1943, p. 105.

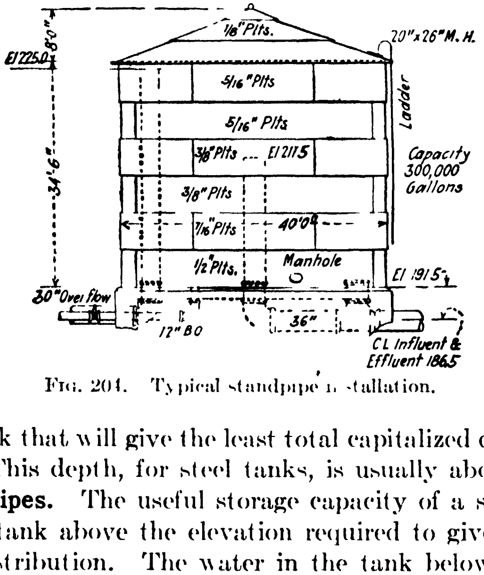

FIG. 204. Typical standpipe installation.

375. Standpipes. The useful storage capacity of a standpipe is the volume of the tank above the elevation required to give the necessary pressure for distribution. The water in the tank below this elevation serves only as a column to support the useful capacity. It is often possible to take advantage of hills or high ground for a standpipe location, with a view toward making the entire capacity of the tank usable. In cities where such sites are not available, the standpipe is not an economical type of reservoir except for the storage of relatively large capacities. Figure 204 shows the typical details of a steel standpipe. The dimensions of a standpipe are determined by the required capacity and height necessary to develop distributing pressures. The cost increases rapidly with the height, because the required strength of the walls, to care for the stresses due to wind and the weight of the water, increases with the square of the height.

The design of a standpipe involves the determination of the economical size to perform the required operating function and an analysis of the effect of the loads upon internal stresses, the stability of the structure, and the foundations. The dead load includes the weight of all permanent construction and fittings. The live load includes the weight of the liquid contents and, for regions where snowfall occurs, a snow load of 25 lb. per sq. ft. of horizontal projection if the roof has a slope of less than 25 deg. with the horizontal. A lateral load due to wind is assumed to be 30 lb. per sq. ft. on a vertical plane surface and 18 lb. per sq. ft. of projected vertical surface when the exposure is a cylindrical surface. The point of application of the wind load is considered to be at the center of gravity of the projected area. In locations subject to unusual winds, 40 and 24 lb. per sq. ft., respectively, should be used instead of 30 and 18. Wind stresses may be neglected if they are less than 25 per cent of those due to dead and live load.

Foundations for standpipes are usually circular, octagonal, or square in plan and are built of masonry. In order that there shall be no tension on the windward side of the foundation, the latter should be proportioned so that the resultant of the vertical forces (weight of foundation and empty tank) and the horizontal force due to wind pass through the middle 25 per cent of the diameter of a circular base, also through the middle 26.5 per cent of the diameter of the inscribed circle of an octagonal base and the middle $33^1/_3$ per cent of the side of a square base. Foundation requirements should be determined from field tests.

Standard references should be consulted for detailed design practice.[1]

376. Materials for Standpipes. Standpipes are usually built of steel or reinforced concrete. Steel is generally the favored material, as experience indicates that it is difficult to make concrete standpipes watertight under heads greater than 50 ft. Concrete has the advantage over steel of lower maintenance cost, when originally well constructed; it is more adaptable to architectural treatment; and it may be erected more quickly. The advantages of steel standpipes over concrete are adaptability to high heads, leaks more easily repaired, slightly lower first cost, greater assurance at the outset of a dependable structure. Steel standpipes are subject to rapid deterioration unless they are painted regularly and otherwise carefully maintained. Copper-bearing steel containing not less than 0.2 per cent copper is recommended for the tank and roof material.

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[1] KETCHUM, M. S., "Structural Engineers' Handbook," Chap. XI, "Steel Standpipes," McGraw-Hill Book Company, Inc., New York, 1924. HOOL and JOHNSON, "Concrete Engineers' Handbook," Sec. 17; "Concrete Standpipes," McGraw-Hill Book Company, Inc., New York, 1918. "Standard Specifications for Riveted Steel Elevated Tanks and Standpipes," *Jour. A.W.W.A.*, Vol. 27, p. 1606, 1935.

377. Steel Standpipes. Steel standpipes are built of curved steel plates welded or riveted together. The diameters of the adjacent courses vary so that the latter will lap inside and outside alternately unless the joints are to be welded. The minimum thickness for side plates is $\frac{1}{4}$ in. for tanks up to 500,000-gal. capacity and $\frac{5}{16}$ in. for larger sizes. The thickness of bottom plates is given by the formula $t = \frac{H + 125}{600}$, in which t is the thickness in inches and H is the height of the standpipe in feet. When the formula gives results between sixteenths, the next higher sixteenth is used. The minimum thickness for bottom plates is $\frac{1}{4}$ in., and the maximum is $\frac{1}{2}$ in. If riveting is used, the diameter, pitch, edge distance, and driving of rivets, as well as calking requirements, should be carefully specified. Minimum diameter of rivets is $\frac{5}{8}$ in. Holes should be tapped, on about 4-ft. centers, through the bottom plates so that grout can be forced through them, after erection, to secure adequate bearing. After grouting, the holes should be plugged with screw plugs. Anchor bolts should not be less than $1\frac{1}{4}$ in. in diameter and should be long enough to provide resistance to uplift due to wind on an empty tank. Bent plates fasten the anchor-bolt casting to the sides of the tank. Steel ladders on the outside provide access to the top of the tank. Inside ladders are seldom required and should never be used in tanks subject to ice formation. Access to the bottom of the tank may be provided by a manhole near the bottom.

378. Cylindrical Concrete Tanks. The side walls of a cylindrical concrete tank may resist bursting by hoop tension or a combination of hoop tension and cantilever bending. The cantilever action is most pronounced immediately above the connection between the vertical walls and the bottom of the tank. Attempts have been made to minimize the cantilever stresses by the construction of a flexible seal between floor and walls, as shown in Fig. 205. The floor is constructed to be independent of the side wall, which can expand and contract without cracking. The lower part of the joint is filled with a mixture of sand, cement, and iron rust. The true seal is formed by a pure gum-rubber strip as shown. Prestressed concrete tanks[1] are built with concrete walls prestressed by tightening steel bands around them to such tension that there is always compression in the tank walls even when filled with water. The steel bands are covered with concrete after stress has been put into them. Advantage claimed for such construction include (1) watertightness, (2) permanence and low upkeep, (3) adaptability to architectural treatment, (4) ease in burying underground, (5) simple and accurate determination of stresses. Disadvantages include relatively high first cost and great weight if used as an elevated tank.

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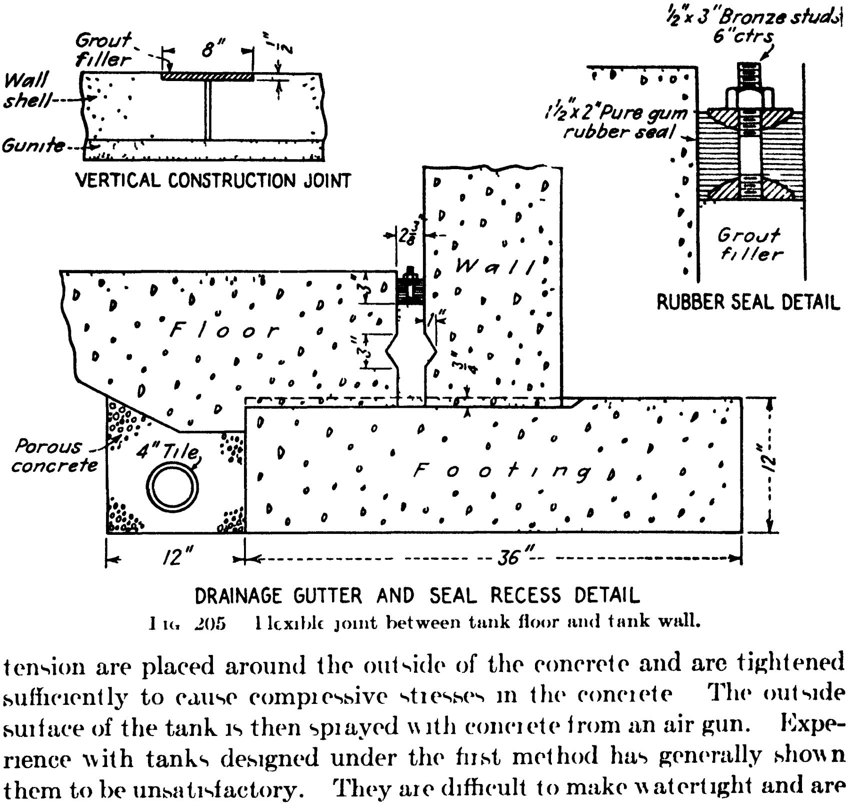
[1] See KENNEDY, R. C., "Design and Construction of Pre-stressed Concrete Tanks," *Jour. A.W.W.A.*, January, 1945, p. 76.

379. Reinforced-concrete Standpipes. Reinforced-concrete standpipes are designed by two different methods: (1) Steel hoops of sufficient cross-sectional area to resist the tension due to the internal water pressure are imbedded in a circular wall built of concrete. (2) A circular concrete wall is built and allowed to set. Steel bands of sufficient cross-sectional area to resist the tension due to the internal water pressure and initial tension are placed around the outside of the concrete and are tightened sufficiently to cause compressive stresses in the concrete. The outside surface of the tank is then sprayed with concrete from an air gun. Experience with tanks designed under the first method has generally shown them to be unsatisfactory. They are difficult to make watertight and are subject to cracking due to temperature changes and to stresses unaccounted for in design. They are also subject to serious damage from frost action. Favorable results have been reported from tanks built by the second method.



DRAINAGE GUTTER AND SEAL RECESS DETAIL

FIG. 205. Flexible joint between tank floor and tank wall.

The principal advantages of reinforced-concrete tanks are rapid erection and adaptability to inexpensive architectural treatment to improve the appearance. Seepage, efflorescence, and spalling, due to frost action, may nullify the last-named advantage.

380. Elevated Tanks. The name "elevated tank" ordinarily refers to the entire structure, consisting of the tank, the tower, and the riser pipe. Elevated tanks are usually cheaper than standpipes, per unit volume of useful capacity, for locations in which pressure requirements necessitate considerable elevation above the surface of the ground. The design of elevated tanks involves the following determinations:

1. The capacity.
2. The required elevation.
3. The size and shape of the structural members required to resist the stresses.
4. Stability of the structure and foundation requirements.
5. The type and installation of appurtenances needed for operation.

The requirements for capacity are stated on pages 327 to 330.

Ample capacity for future needs should be provided. The required elevation depends upon the desired pressure at the most remote part of the distributing area served by the tank. Features of the structural members are discussed in a general way in the following paragraphs. The student is referred to standard texts[1] for detailed methods of elevated-tank design.

The design of the foundation for an elevated tank is important. Unequal settlement causes severe strains on the various members and may cause serious leaks. An exploration of subsurface conditions is made for large structures. The bearing power of the foundation soil should be determined by tests.

Towers supporting elevated tanks are designed to act as a unit with the tank and, therefore, to resist the combined stresses due to weight and bending. Care must be taken to provide for uplift on the windward side, with suitable foundations and proper anchorage. Earthquake stresses are sometimes provided for.[2]

381. Elevated Tanks of Steel. Tentative Standard Specifications for Elevated Steel Tanks, Standpipes, and Reservoirs have been prepared by the American Water Works Association.[3] Tentative Recommended Practice for the Repainting of Elevated Steel Tanks and Water Storage Tanks, with notes on repairs, have been published by the American Water Works Association.[4] Abstracts from these specifications are shown in Table 64. The thickness of the steel shell of the tank is fixed by the hydrostatic pressure to which the plates are subjected, by the allowable unit stresses, and by the joint efficiencies. The trend in practice is toward welding. Nearly all tanks over 500,000 gal. capacity are welded, and most small tanks are now of welded construction.

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[1] See footnote on page 341.

[2] See also RUGE, A. C., "Earthquake Resistance of Elevated Water Tanks," *Trans. A.S.C.E.*, Vol. 103, p. 889, 1938.

[3] See *Jour. A.W.W.A.*, December, 1940.

[4] See *Jour. A.W.W.A.*, July, 1943.

TABLE 64.—EXTRACTS FROM TENTATIVE STANDARDS SPECIFICATIONS FOR ELEVATED STEEL WATER TANKS, STANDPIPES, AND RESERVOIRS[1]

Loads: Dead load is weight of all permanent construction and fittings, with steel at 490 lb. per cu. ft. and concrete at 144 lb. per cu. ft. *Live load* is the weight of all liquid in the tank. *Snow load* is 25 lb. per sq. ft. of horizontal projection of surfaces having slope of less than 30 deg. with the horizontal. *Wind load* 30 lb. per sq.ft. on vertical plane surface, 18 lb. per sq. ft. on projected areas of cylindrical surfaces, and 15 lb. per sq. ft. on projected areas of conical and double curved plate surfaces. *Earthquake load* (to be specified by purchaser). *Balcony and ladder,* 1,000 lb. at any point in balcony, 500 lb. at any point in roof, and 350 lb. on each section of ladder.

Unit stresses (in pounds per square inch): Structural steel, tension 15,000; compression, 15,000; shear (webs) 9,750.

Roofs: All tanks storing drinking water should have roofs.

Capacities: For *standpipes and reservoirs* (in thousands of gallons): 50-60-75-100-150-200-250-300-400-500-750-1,000-1,500-2,000-2,500-3,000-4,000. For *elevated tanks:* 5-10-15-20-25-30-40-50-60-75-100-150-200-250-300-400-500-750-1,000-,1500-2,000-2,500.

Shell height: For standpipes and elevated tanks: from 20 to 50 ft. by even 2-ft. intervals. From 50 to 100 ft. by even 5-ft. intervals. From 100 to 200 ft. by even 10-ft. intervals.

Diameters of reservoirs: same as heights for standpipes and elevated tanks.

Accessories: Standpipes, reservoirs, and elevated tanks: overflow, outside tank ladder, roof ladder, roof hatch, vent. *Standpipes and reservoirs:* shell manhole, pipe connection. *Elevated tanks:* tower ladder, roof finial, steel riser pipe, and pipe connection.

[1] See *Jour. A.W.W.A.,* December, 1940.

Towers for steel tanks are usually built of steel but may be built of reinforced concrete or even wood. The posts are designed to resist the combined stresses due to direct load and moment due to wind. The towers may be divided, by horizontal struts, into stories from 10 ft. to 12 ft. 6 in. in height. Minimum thickness of tower metal is ¼ in. except in webs of channels and I beams and in other minor parts that carry little or no load. The thickness of cover plates in columns should be one-fiftieth of the distance between rivet lines but not less than ¼ in. Diagonal bracing is used between posts. Figure 206 shows typical details of a steel tower.

An ideal bottom shape for minimum amount of steel would be a shape that put only tension in the steel. Practical considerations prevent the fabrication of such a shape, and the changes in depth of water in a tank make its design impossible. Flat and conical bottoms accentuate flexural stresses.[1] Hemispherical, ellipsoidal, and radial-cone shaped bottoms more nearly approach the ideal and are more commonly used. The last two types make it possible to increase the diameter for a given capacity, as compared with a hemispherical bottom tank, so that the pumping head and pressure variations may be minimized.

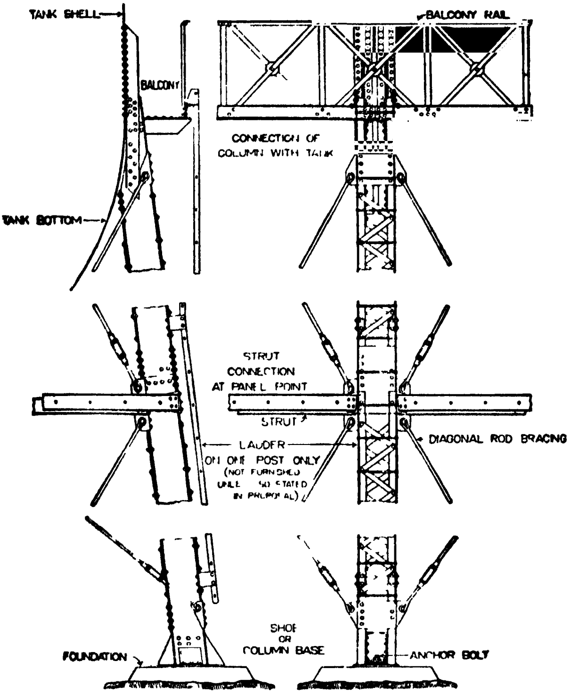
TANK SHELL
BALCONY RAIL
BALCONY
CONNECTION OF COLUMN WITH TANK
TANK BOTTOM
STRUT CONNECTION AT PANEL POINT
STRUT
LADDER ON ONE POST ONLY (NOT FURNISHED UNLESS SO STATED IN PROPOSAL)
DIAGONAL ROD BRACING
SHOE OR COLUMN BASE
FOUNDATION
ANCHOR BOLT
SIDE VIEW
VIEW LOOKING TOWARD CENTER OF TOWER

FIG. 206. Typical tower details for elevated water tanks. (*Courtesy Chicago Bridge and Iron Works.*)

Standard tanks are available for a maximum capacity of 1 million gal. in the hemispherical-bottom type, 2 million, in the ellipsoidal-bottom type. The first type is limited to a diameter of about 50 ft. and the second, to about 90 ft.

The hemispherical bottom is widely used for small tanks, but its range in head for a given capacity is greater than for the other types, since its diameter is limited to about 50 ft. Its shape affords low and easily calculable stresses. The radial and circumferential stresses are equal to one-half the stresses in a cylinder of like radius, subject to the same hydrostatic pressure.

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[1] See also LEE, C. A., "Stresses in Conical Bottoms of Tanks," *Eng. News-Record*, May 4, 1944, p. 100.

The upper limit of diameter for an ellipsoidal-bottom tank is 80 to 90 ft. In this type of tank the riser pipe is used to support part of the weight of the tank and water. The bottom of the tank acts as a diaphragm and eliminates the necessity for an expansion joint between the tank and the riser pipe. Figure 207 shows a typical detail of a tank with ellipsoidal bottom.

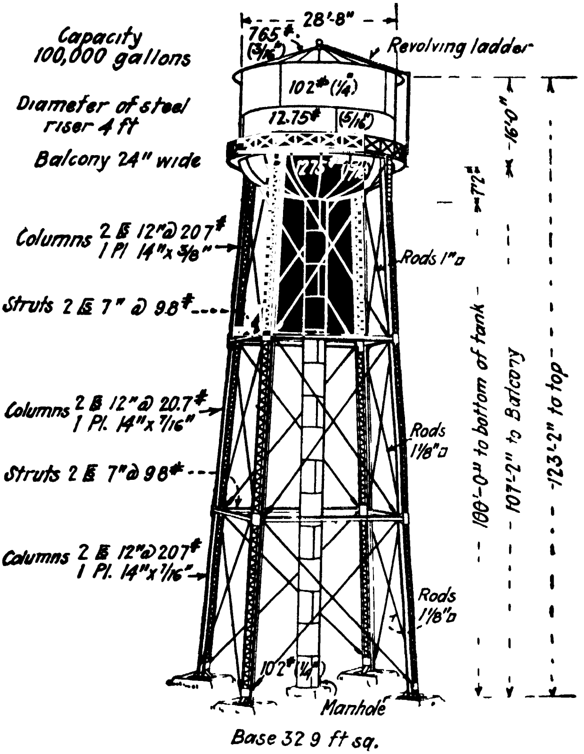

FIG. 207—Typical elevated steel tank with ellipsoidal bottom

The radial-cone and toroidal types make it possible to develop very large capacities with low ranges of head. One tank has been constructed that has a capacity of 11,250,000 gal. with a head difference of only 16 ft.[1] Figure 208 is a photograph of a modern steel elevated tank having a radial-cone bottom. Elevated tanks of steel must be painted regularly to avoid rapid deterioration.

382. Appurtenances for Steel Tanks. *Roofs.* The roof of an elevated tank or standpipe may be considered as an appurtenance, since it may or may not be used. In most cases the roof provides stiffening for the top of the tank. Roofs are ordinarily used as a protection against temperature changes and contamination caused by birds and flying insects. They may be omitted in temperate climates and in locations relatively free from contamination.

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[1] See also JACKSON, J O, "Elevated Tanks for Distribution Systems," *Jour. A W W A*, Vol 23, p 1743, 1931

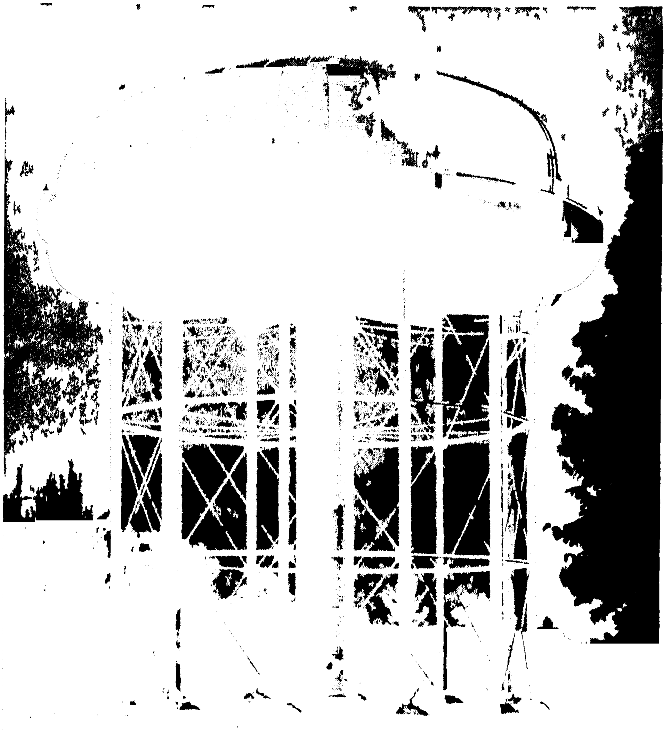

Fig. [illegible] Modern radial-cone-bottom elevated tank (*Courtesy of Chicago Bridge and Iron Works*)

Spider Rods. Spider rods are rods radiating from the center of the tank, in a horizontal plane, and are attached to and supported only by the wall of the tank. The rods may be used[1] (1) to hold the tank round during erection; (2) to support scaffolding and permanent interior parts, such as pipes; and (3) to prevent collapse of the shell by wind when the tank is empty. The rods are objectionable in cold climates because they may cause collapse of the tank due to ice forming on them In cold climates where ice may form in the tank, there should be no obstruction on the inside of the tank to prevent the rise and fall of the ice cap on the water surface.

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[1] See also PERRYMAN, J M, discussion, *Water Works & Sewerage*, September, 1943, p 327

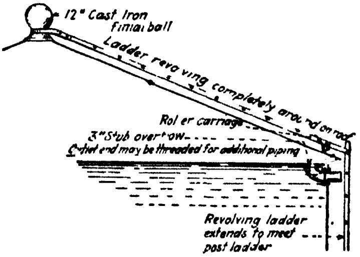
12" Cast Iron finial ball
Ladder revolving completely around on roof
Roller carriage
3" Stub overflow
Outlet end may be threaded for additional piping
Revolving ladder extends to meet post ladder

Fig. 209.-- Revolving ladder for elevated tanks.

Ladders. Ladders are supplied for standpipes and elevated tanks to give access from a point 12 ft. above the concrete foundation to the top of the tank. On tanks without roofs the ladder should extend to a point 3 ft. above the shell. On tanks with roofs a revolving ladder fastened at the finial in the peak is usually used. The lower end of the ladder is equipped so that a boatswain's chair may be attached to give access to all parts of the shell. Revolving ladders for elevated tanks may extend from the peak to the balcony, in which case the fixed ladder from the balcony to top of the tank is omitted. Figure 209 shows the general arrangement of the revolving ladder. Note the ladder in Fig. 208.

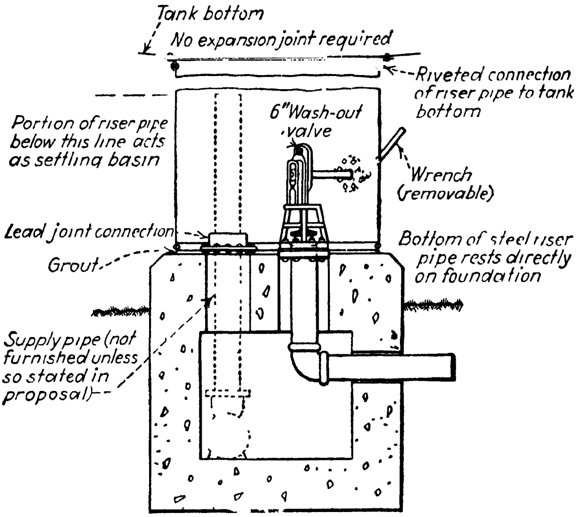
Tank bottom
No expansion joint required
Riveted connection of riser pipe to tank bottom
Portion of riser pipe below this line acts as settling basin
6" Wash-out valve
Wrench (removable)
Lead joint connection
Bottom of steel riser pipe rests directly on foundation
Grout
Supply pipe (not furnished unless so stated in proposal)

Fig. 210. Detail at base of riser pipe of ellipsoidal-bottom elevated tank.

Balcony and Handrail. The width of the balcony should be at least 24 in. for 15-ft. diameter tanks and not less than 30 in. for larger tanks The floor should have a minimum thickness of $^5/_{16}$ in. and be suitably punched for drainage. The handrail is at least 3 ft. high and is constructed so as to stiffen the outer chord of the girder. The balcony support forms a part of a circular girder to strengthen the connection between the side walls and bottom.

Riser Pipe. In tanks having hemispherical bottoms, the riser pipe is usually of cast iron and may or may not be provided with a frostproof covering. An expansion joint between tank and riser pipe is required. In ellipsoidal-bottom tanks, the riser pipe is of steel and serves to support part of the weight of tank and its contents. The diameter of the riser pipe is ordinarily 3 to 4 ft. for tanks up to 150,000 gal. and 5 to 10 ft. for larger tanks. The expansion joint and frostproof cover are omitted. A supply pipe, as shown by the dotted lines in Fig. 210, admits the water to the riser pipe. A washout valve serves to remove the sediment that collects in the lower portion. Recommended sizes of riser pipes are listed in Table 65.[1]

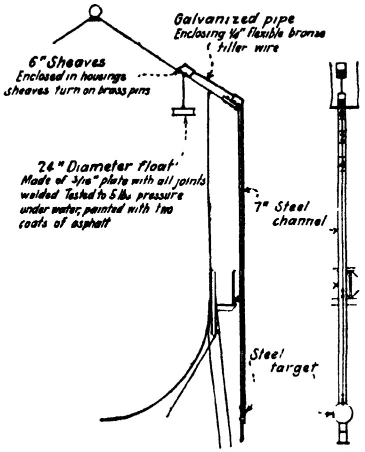
Galvanized pipe
Enclosing 1/8" flexible bronze tiller wire
6" Sheaves
Enclosed in housings sheaves turn on brass pins
24" Diameter float
Made of 3/16" plate with all joints welded Tested to 5 lbs pressure under water, painted with two coats of asphalt
7" Steel channel
Steel target

FIG. 211. Standard indicator for elevated steel tanks.

Water-lever Indicators. A device to indicate the level of water in an elevated tank or a storage reservoir is essential to its successful operation. Such devices vary from a simple float and stick subject to direct observation, to ingenious telephone and radio remote-control devices that

TABLE 65.—SIZES OF RISER PIPES AND CONNECTING PIPES FOR ELEVATED TANKS[1]

Capacity of tank, gal	Delivery rate for one-half capacity, g p m.	Size of connecting main, in.
100,000	166[2]	6
200,000	333[2]	8
500,000	417	8
1,000,000	833	12
2,000,000	1,666	2 @ 12 or 1 @ 16
5,000,000	4,165	2 @ 16 or 1 @ 20

[1] From BOOTH, G. M., "Practical Studies of Distribution Systems," *Jour. A.W.W.A.*, September, 1944, pp. 957 and 963.

[2] For 5-hr. duration. Others for 10-hr. duration.

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[1] From BOOTH, G. M., "Practical Studies of Distribution Systems," *Jour. A.W. W.A.*, September, 1944, pp. 957 and 963.

transmit information concerning water levels to any desired distance and do it continuously or on demand.[1]

Overflow. Overflow of the tank is provided for by connecting a conductor pipe to a pipe through the wall of the tank or to a trough extending around the outside of the tank. Figure 209 shows one method of caring for overflow.

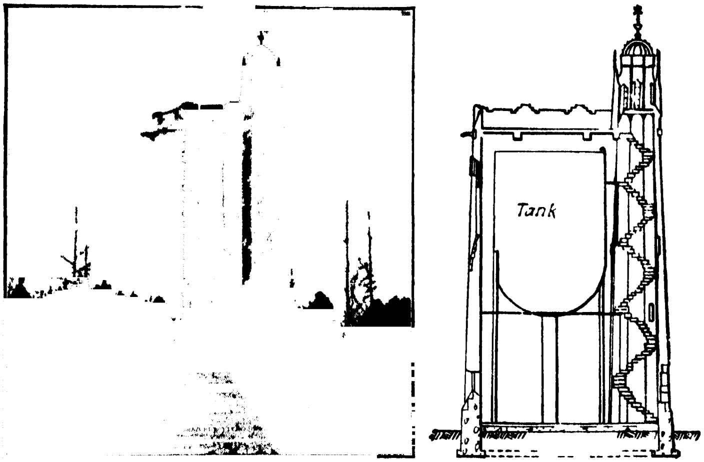

Fig. 212.—Elevated tank at Gate of Heaven Cemetery Designed by Charles Wellford Leavitt and Son (*Courtesy of Chicago Bridge and Iron Works*)

383. Architectural Treatment. The old-fashioned steel tanks are usually unsightly. Because of this fact, their use in certain locations is objectionable. Considerable study is now being directed toward improving the appearance of steel tanks, and the results have justified the effort as evidenced by Fig. 208. With proper attention to details of towers, balconies, and general lines and proportions, steel tanks may be made quite pleasing to the eye. In places where a steel tank cannot be made to harmonize with the surroundings, they may be covered with masonry. Such a treatment is shown in Fig. 212.

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[1] See also *Eng. and Contr*, Vol 44, p 360, 1915; *Power*, Vol 38, p. 888, 1913; Wertz, C F, "Telemarking," *Water Works & Sewerage*, May, 1946, p 182, Colman, E. A., and E. L Hamilton, "The San Dimas Water-stage Transmitter," *Civil Eng.*, June, 1944, p. 257.

CHAPTER XXII

DESIGN OF DISTRIBUTION SYSTEMS

384. Description of Distribution Systems. A waterworks-distribution system includes pipes, valves, hydrants, and appurtenances for conveying water; reservoirs for storage, equalizing, and distribution purposes; service pipes to the consumers; meters; and all other parts of the conveying system after the water leaves the main pumping station or the main distribution reservoirs.

The layout of distribution systems may be classified, for convenience, as (1) circle, or belt, systems, (2) gridiron systems, and (3) tree systems. The names are descriptive of the manner in which the distribution pipes are laid out on the plan. Most distribution systems contain features of each type of layout; practically none is an example of one system alone, although the gridiron system is probably the most common, particularly in large cities.

In cities where there are differences in surface elevation of more than about 200 ft., the distribution system is usually divided into zones in order to avoid excessive pressures in the lower zones. The different zones are usually supplied by independent pumps or reservoirs, which are usually characterized by the terms "high service" and "low service."

Some cities are supplied from two sources, one pure for domestic use and one impure for fire protection or industrial purposes. The practice is universally condemned if the two supplies are interconnected or if there is any opportunity for use of the impure supply for personal needs or in the preparation of foods or beverages. The use of an impure supply should be limited exclusively to fire protection, boiler water, cooling, and similar purposes.

385. Design of a Distribution System. The adequacy of a distribution system is determined by the adequacy of the pressures that exist at various points in the system under actual or assumed conditions of operation. On the one hand, the pressures must be sufficient to serve the consumers or the fire demand. On the other hand, pressures that are unnecessarily high require an unjustifiable expenditure for safety and maintenance.

There is no direct method of design for distribution systems or of determining the size of pipes required. In a new system the pipe sizes must be assumed, and the system investigated for the pressure conditions that will result from various requirements of demand. These can be studied by the construction of pressure contours, as shown in Fig. 214. The pipe sizes are then changed, if necessary, and the system reinvestigated until satisfactory conditions are indicated. In existing systems, the pipe sizes are known, so that only one investigation for each demand requirement is necessary. But when changes are contemplated, the sizes of the new pipes must be treated the same as in the design of a new system.

The factors that afford loss of head or pressure are interdependent and include size of pipe, quantity of flow, and hydraulic losses due to friction and other causes. Usually, losses due to friction in straight pipe only are considered. If it is desired to include minor losses, it is most convenient to account for their effects by adding a length of straight pipe in which the friction loss is equivalent to the calculated minor loss.

The friction loss in many pipes increases on account of the corrosion and deposition that may occur with use. Since it is usually presumed that a permanent municipal distribution system will be used over a long period of years, it becomes necessary for the engineer to anticipate the future and design the system for the condition that will obtain near the end of the time when the amounts set aside for depreciation will have returned the first cost. For cast-iron pipe a coefficient of $C = 100$ in the Hazen and Williams formula is ordinarily used for original calculation. Table 6 (page 35) gives formulas for calculating the effect of time on friction loss for steel pipe.

The design or investigation process may generally be considered to contain the following steps:

1. Prepare, on a map of the city, a layout showing the lengths and sizes of all pipes of the proposed or existing system. This layout should show also all the hydrants, valves, elevated tanks, and other appurtenances proposed or existing for the operation of the system.
2. Indicate the known or assumed demands that are or will be required at important points in the system.
3. Calculate the pressure drop that will occur between the source of supply and the critical points in the system.
4. Revise the pipe sizes, and repeat step 3 until satisfactory pressures for the required demands are indicated for all points in the system. Further discussion of the technique of analysis will be found in the succeeding section.

386. Selection of Pipe Sizes. Preliminary estimates for the sizes of pipes in a distribution system are frequently based on experience. When the rate of flow is known, the diameter can be determined by assuming a velocity of 3 to 4 f.p.s. The rules for minimum size of pipe are frequently violated to reduce the first cost of pipe, without proper consideration of over-all economy and of pressure losses.

In the selection of the sizes of pipes, some of the following principles may serve as a guide:

1. Use no pipes less than 4 in. in diameter, and use these sparingly. Information in Table 12 shows the 4-in. pipe to be relatively costly per unit of capacity when compared with larger pipes. The National Board of Fire Underwriters recommends a minimum size of 8 in., but permits 6-in. pipe in gridirons when the length between connections is not over 600 ft. The percentages of sizes of pipes in various systems are shown in Table 66.
2. Place fire hydrants on pipes 6 in. or larger in diameter, and no hydrants should be placed on long dead ends 6 in. or smaller in size.
3. Pipes should, in general, be interconnected at intervals closer than about 1,200 ft.
4. It is better to lay two moderately large mains parallel on streets three or four blocks apart than to lay one large main, even with slightly greater capacity, down one street.
5. Arterial mains should be in duplicate but not in the same street.
6. When railroads, streams, or other barriers lie between water source and parts of the city, not less than two and preferably more well-separated arterial pipes should cross the barrier.
7. Emergency interconnections that can be quickly made between safe potable supplies should be provided where available.

TABLE 66.—DISTRIBUTION OF PIPE SIZES IN EIGHT WATER-DISTRIBUTION SYSTEMS[1]

Size of pipe, in.	4	6	8	10	12	14	16	18	Others
Per cent total length of pipes	15	55	15	..	7.5				[2]
	16.5[4]	49.1	11.2	4.3	8.1	2.2	2.6	1.2	4.6[3]

[1] The weighted average of mains in most distribution systems has been found to be close to 7 in.

[2] From HOWSON, L. R., "The Distribution System," *Jour. A.W.W.A.*, December, 1943, p. 1517.

[3] From "Selected Canadian Municipalities," *Jour. A.W.W.A.*, April, 1945, p. 397.

[4] Includes 4- and 5-in. pipes.

When a preliminary layout of the distribution system is complete, assumptions are made concerning the rates of flow into and from the system at strategic points and the system is analyzed for head losses. If head losses are found to be unsatisfactory, other sizes of pipe are chosen and the system is reanalyzed for head losses.

387. Analyses of Pressures in Distribution Systems.[1] Methods that have been used in the analyses of distribution systems include[2] (1) the equivalent pipe method, omitting and combining pipes to form series and parallel compound pipes;[1] (2) graphical methods, described by Freeman,[2] by Kingsbury,[3] and by Palsgrove[4] and by Rensselaer Polytechnic Institute,[5] (3) the construction and analysis of an electrical analogy, as described by Camp and Hazen;[6] and (4) the Hardy Cross method of successive approximations.[7] Because of the relative simplicity and accuracy of the Hardy Cross method, it is used almost exclusively.

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[1] See also CAMP, T. R., "Hydraulics of Distribution Systems—Some Recent Developments in Methods of Analysis," *Jour. New Engl. Water Works Assoc.*, 1943, p. 334.

[2] See also GAVETT, WESTON, "Computation of Flows in Distribution Systems," *Jour. A.W.W.A.*, March, 1943, p. 267.

358. Hardy Cross Method of Analysis.[8] The Hardy Cross method of network analysis is based on the fact that the loss of head from one point to another in a distribution system is the same, regardless of the path followed. In order to find the correct flow in the pipes of a network distribution system, any convenient flows are assumed and the head losses between points in the distribution system are computed with the aid of any desired pipe-flow formula, using the rates of flow just assumed. If the head losses by different paths between two points are not equal, the assumed rates of flow are corrected by ΔQ, computed from the expression[9]

$$\Delta Q = \frac{\Sigma rQ^n}{\Sigma nrQ^{n-1}} \tag{1}$$

in which the numerator is the algebraic sum and the denominator is the arithmetic sum (*i.e.*, the sum without regard to signs). If r represents the relative resistance and n is the exponent of Q in the typical flow formula $h = rQ^n$, it can be shown that, in the Hazen and Williams formula, $r = \frac{K}{C^{1.85}d^{1.86}}$. The proof is as follows:

In the Hazen and Williams formula

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[1] See also HOWLAND, W. E., and FARR, F., JR., "Computation of Equivalent Lengths of Parallel Pipe Systems," *Jour. A.W.W.A.*, Vol. 33, p. 237, 1941.

[2] FREEMAN, J. R., "Arrangement of Hydrants and Water Pipes for Fire Protection," *New Engl. Water Works Assoc.*, Vol. 7, pp. 49, 152, 1892–1893.

[3] KINGSBURY, F. H., "Capacity of Twin Pipe Line," *Eng. News*, Vol. 76, p. 1239, 1916.

[4] PALSGROVE, G. K., "Graphical Analysis of a Dual Flow Water Supply System," *Eng. News-Record*, Vol. 107, p. 422, 1931.

[5] *Bull.* 37, August, 1932.

[6] CAMP, T. R., and H. L. HAZEN, "Hydraulic Analysis of Water Distribution Systems by Means of an Electrical Network Analyzer," *Jour. New Engl. Water Works Assoc.*, December, 1934, p. 383.

[7] CROSS, HARDY, "Analysis of Flow in Networks of Conduits and Conductors," *Univ. Ill. Expt. Sta. Bull.* 286, 1936; and DOLAND, J. J., "Simplified Analysis of Flow in Distribution Systems," *Eng. News-Record*, Vol. 117, p. 475, 1936.

[8] See also HOWLAND, W. E., and FARR, F., JR., "Balancing Head Losses in a Pipe Network," *Eng. News-Record*, Feb. 27, 1941, p. 50.

[9] A nomograph for computing values of this expression is given by G. M. Slight in *Eng. News-Record*, Jan. 1, 1942, p. 61.

$$S = \frac{h}{l} = \frac{3.030V^{1.85}}{C^{1.85}D^{1.17}}$$

where the linear units are in feet. But

$$V = \frac{Q}{A} = \frac{4Q}{\pi d^2}$$

Substituting this value for V in the above expression, using the constant K to care for the units, and allowing length to be one unit

$$r = \frac{K}{C^{1.85}d^{4.86}} \tag{2}$$

Values of r are given in Table 67, based on the resistance of a 12-in. pipe, of unit length, with $C = 110$ as unity.

TABLE 67. RELATIVE RESISTANCES OF CIRCULAR PIPES BY HAZEN AND WILLIAMS FORMULA

Diameter, in	$C = 90$	$C = 100$	$C = 110$	$C = 120$	$C = 130$	$C = 140$
4	300	250	210	181	156	135
6	40	35	29.2	25	22	19
8	10	8.6	7.2	6.2	5.3	4.6
10	3.4	2.9	2.4	2.1	1.8	1.55
12	1.13	1.2	1	0.86	0.71	0.645
14	0.67	0.56	0.47	0.40	0.35	0.30
16	0.36	0.30	0.25	0.22	0.185	0.16
18	0.20	0.17	0.14	0.12	0.10	0.09
20	0.10	0.08	0.07	0.06	0.05	0.045
24	0.04	0.036	0.03	0.026	0.022	0.019
30	0.0161	0.0138	0.0115	0.010	0.0085	0.0074
36	0.0069	0.0058	0.0048	0.0041	0.0036	0.0031

The analysis is repeated as often as necessary until ΔQ becomes as small as desired.

Example: Let it be desired to determine the head losses, in feet, and to draw the 1-ft. pressure contours for the conditions shown in the pipe network in Fig 213.[1]

Solution: Essential computations are shown in Table 68. Columns 1, 2, 3, 4, and 5 are data.

The first step is to complete lines 1 to 4 in columns 6 to 15.

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[1] This network was used by G. M. Fair in a simplification of the Hardy Cross method published in *Eng. News-Record*, Mar. 3, 1938, p. 342, and in subsequent discussions and simplifications: W. E. Howland and F. Farr, Jr., *Eng. News-Record*, Feb. 27, 1941, p. 50; and by J. A. Conklin, *Eng. News-Record*, Sept. 11, 1941, p. 96.

TABLE 68.—COMPUTATIONS OF HEAD LOSSES BY HARDY CROSS METHOD FOR PIPES SHOWN IN FIG. 213

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Circuit	Pipe number	Length, 1,000's ft	Diameter, inches	Coefficient C	r = relative resistance	Q (by Conklin)[1]	Q_1 (correction)	$Q^{0.85}$	$Q^{1.85}$	$rQ^{0.85}$	$rQ^{1.85}$	Δh	ΔQ	Q_1	Q_1 (correction)	$Q_1^{0.85}$	$Q_1^{1.85}$	$rQ_1^{0.85}$	$rQ_1^{1.85}$	Δh	ΔQ	Q	Q (correction)	$Q^{0.85}$	$Q^{1.85}$	$rQ^{0.85}$	$rQ^{1.85}$	Δh	ΔQ	Q in c.f.s.	h in ft
I 1	1	2	12	120	1.7	+57.9	57.9	31.5	1825	53	3103			57.7	57.7	34.8	2010	59	3417			57.5	57.5	34.8	2000	591	3400			0.885	1.09
2	2	0.5	6	120	12.0	+28.95	+28.95	17.5	508	210	6096	−176	−0.2	28.8	28.8	17.4	501	209	6012	+205	−0.2	28.6	28.9	17.4	503	209	6040	+130	−.047	0.445	2.22
3	3	0.5	8	100	4.2	−42.1	42.1	23.9	1006	101	4235			42.3	42.3	24.1	1020	101	4284			42.5	42.5	24.1	1025	1013	4310			0.655	1.56
4	4	2	8	100	16.8	−21.3	21.3	13.4	285	226	4788			21.5	21.6	13.6	294	228	4940			21.8	21.8	13.7	298	230	5000			0.336	1.82
5																															
II 6	5	0.5	6	120	12.0	+28.95	+28.95	17.5	508	210	6100			29.0	29.0	17.5	508	210	6100			28.7	28.7	17.3	497	208	5960			0.442	2.22
7	6	0.5	6	120	12.0	+19.0	19.0	12.2	232	146	2730	+30	0	19.0	19.0	12.2	232	146	2780	+405	−0.3	18.7	18.7	12.05	225	145	2700	+40	−.034	0.288	0.94
8	2	0.5	6	120	12.0	−28.95	28.8	17.4	501	209	6030			28.8	28.6	17.3	495	208	5940			28.9	28.9	17.4	503	209	6040			0.445	2.22
9	7	0.5	8	100	4.2	−33.8	33.8	19.8	670	83	2820			33.8	31.9	18.9	603	79	2535			32.2	32.2	19.1	615	80	2580			0.496	0.93
10																															
III 11	4	2	8	100	16.8	+21.3	21.5	13.6	292	229	4900			21.6	21.8	13.7	299	230	5030			21.7	21.8	13.7	298	230	5005			0.336	1.82
12	8	0.5	6	120	12.0	+16.5	16.5	10.8	178	130	2140			16.6	16.6	10.8	179	130	2170	+110	−0.07	16.5	16.5	10.8	178	130	2140			0.254	0.78
13	9	0.5	6	120	12.0	−20.8	20.8	13.2	274	158	3290	−160	+0.1	20.7	20.7	13.1	271	157	3250			20.8	20.8	13.2	274	158	3290	−55	+.034	0.321	1.12
14	10	2	6	120	48.0	−10.8	10.8	7.5	81	362	3910			10.7	10.7	7.5	80	362	3840			10.8	10.8	7.5	81	362	3910			0.167	1.49
15																															
IV 16	7	0.5	8	100	4.2	+33.8	33.8	19.8	670	83	2810			31.9	32.2	19.1	615	80	2580			31.8	32.2	19.1	615	80	2580			0.496	0.93
17	11	0.5	8	100	4.2	+52.8	52.8	30.2	1598	127	6710	−1920	−1.9	50.9	50.9	28.5	1451	119	6095	380	−0.4	50.5	50.5	28.0	1414	118	5940	+90	−.09	0.777	2.13
18	8	0.5	6	120	12.0	−16.5	16.6	10.8	179	130	2150			18.5	16.5	10.8	178	130	2135			16.9	16.5	10.8	178	130	2140			0.254	0.78
19	12	0.5	6	120	12.0	−27.3	27.3	16.6	454	199	5490			29.2	29.2	17.6	514	211	6160			29.6	29.6	17.7	524	212	6290			0.455	2.29

[1] Computations taken from *Eng. News-Record*, Sept. 11, 1941, p. 96.

Column 6 is computed from the expression

$$r = \frac{K}{C^{1.85}d^{4.86}}$$

Solutions of this expression are given in Table 67.

Column 7 may be assumed as any convenient percentage of the total appropriate to the principle that total flow into an intersection is equal to total flow from that intersection; or it may be computed by Conklin's method, explained in Sec. 389. In this example it has been computed by Conklin's method. In any circuit the plus sign indicates that the direction of flow is from the point of assumed inlet into the circuit, to the point of assumed outlet from the circuit. The minus sign indicates the reverse direction of flow in the circuit. The aim is to balance the plus and the minus head losses.

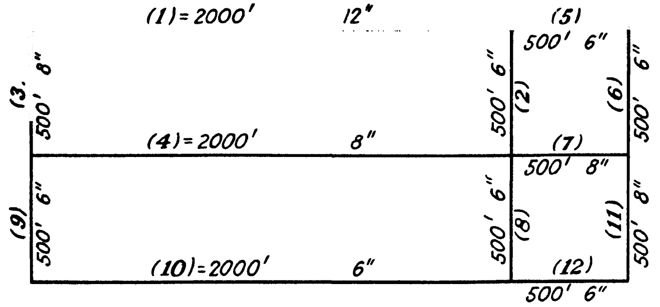

FIG. 213.– Pipe network for Cross method of computation of distribution losses. (*Layout originally selected by G. M. Fair, Eng. News-Record, Feb. 27, 1941, p. 50.*)

TABLE 69.—VALUES OF THE 0.85 POWER OF NUMBERS

N	0	1	2	3	4	5	6	7	8	9
0	0	1.0	1.8	2.5	3.2	3.9	4.6	5.2	5.9	6.5
10	7.1	7.7	8.3	8.9	9.5	10.0	10.6	11.1	11.6	12.2
20	12.8	13.3	13.8	14.4	14.9	15.4	15.9	16.4	16.9	17.5
30	18.0	18.5	19.0	19.5	20.0	20.5	21.0	21.5	22.0	22.5
40	23.0	23.4	23.9	24.3	24.8	25.3	25.8	26.3	26.8	27.3
50	27.8	28.2	28.7	29.1	29.6	30.0	30.5	31.0	31.4	31.9
60	32.4	32.9	33.3	33.8	34.2	34.7	35.1	35.6	36.0	36.5
70	37.0	37.4	37.9	38.3	38.7	39.1	39.6	40.0	40.5	41.0
80	41.5	42.0	42.4	42.8	43.3	43.7	44.1	44.5	45.0	45.4
90	45.8	46.3	46.7	47.1	47.6	48.0	48.4	48.8	49.2	49.6

Column 8. This figure corresponds with the last preceding corrected value in the table for this particular length of pipe. There are no corrections to be applied in lines 1 to 4.

Column 9 is column 8 raised to the 0.85 power. Values are given in Table 69.

Column 10 is the product of columns 8 and 9.

Column 11 is the product of columns 6 and 9.

Column 12 is the product of columns 6 and 10.

Column 13 is the difference between the sum of lines 1 and 2 and the sum of lines 3 and 4 in column 12.

Column 14 is computed from formula (1) on page 355. It is column 13 divided by the sum of lines 1 to 4 in column 11, disregarding sign, multiplied by n.

Column 15 gives corrections applied to column 8.

The next step is to complete lines 6 to 9 in columns 7 to 15. The procedure is as before except that the figure entered in column 8 is the latest preceding corrected value in the table. For example, in line 8 the corrected figure 28.8 is inserted to correspond with the correction in line 2, column 15.

A similar procedure is followed until the values of ΔQ are as small as desired, in this case less than 0.1 per cent. Values of rate of flow are computed in cubic feet per second and are entered in column 31; head losses are computed by the Hazen and Williams formula and are entered in column 32.

The 1-ft. pressure contours are then drawn, as indicated in Fig. 214.

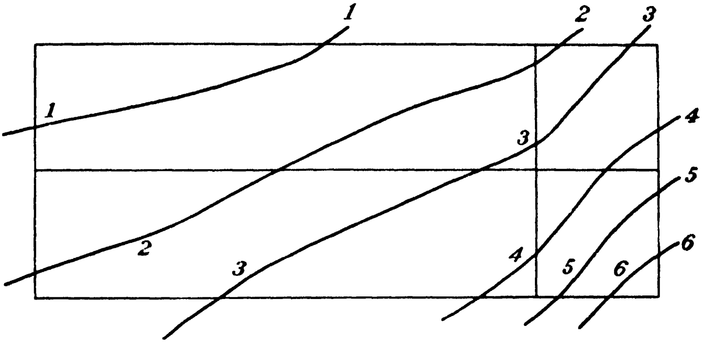

FIG. 214.—Pipe network showing piezometric contours, 1-ft. intervals.

389. Assumptions of Rates of Flow. Good judgment in the assumptions of rates of flow in a distribution system may diminish the computations required in a complete analysis of the flow distribution. Judgment may be guided by the approximate relationship

$$\frac{Q_1}{Q_1 + Q_2} = \frac{r_2^{1/2}}{r_1^{1/2} + r_2^{1/2}} \tag{3}$$

in which Q_1 represents the rate of flow in one pipe taking water away from a junction, Q_2 represents the rate of flow in the other of two pipes taking water away from the junction, and r_1 and r_2 represent, respectively, the relative resistances in each of the two pipes.

The relationship can be proved as follows:

It is known from the Chézy formula that

$$Q_n = \frac{kd_n^{1/2}}{l_n^{1/2}} \tag{4}$$

where Q_n = rate of flow in a pipe
d_n = its diameter
l_n = its length
k = a constant

It is shown in formula (2) that, based on the Hazen and Williams formula for a unit length

$$r = \frac{F}{d^{4.86}} \tag{5}$$

in which F is a constant. It can be shown that for the Chézy formula

$$r = \frac{Fl}{d^5} \tag{6}$$

Then, substituting Eq. (6) in Eq. (4) we have

$$Q_n = \frac{K}{r_n^{\frac{1}{2}}} \tag{7}$$

Hence,

$$\frac{Q_1}{Q_1 + Q_2} = \frac{r_2^{\frac{1}{2}}}{r_1^{\frac{1}{2}} + r_2^{\frac{1}{2}}}$$

It can be shown similarly that

$$\frac{Q_1}{Q_1 + Q_2 + Q_3} = \frac{r_2^{\frac{1}{2}} r_3^{\frac{1}{2}}}{r_1^{\frac{1}{2}} r_2^{\frac{1}{2}} + r_1^{\frac{1}{2}} r_3^{\frac{1}{2}} + r_2^{\frac{1}{2}} r_3^{\frac{1}{2}}} \tag{8}$$

where the subscripts indicate the flows and resistances, respectively, of three pipes taking water away from a junction.

The solution of the following problem illustrates the application of formula (3) to the distribution of flow in a pipe network.

Problem: If 100 g.p.m. is entering a junction from which the flow is carried away by pipe *A* with a relative resistance of 9, and by pipe *B* with a relative resistance of 4, what is the rate of flow in each pipe?

Solution:

$$\frac{Q_A}{Q_A + Q_B} = \frac{r_B^{\frac{1}{2}}}{r_A^{\frac{1}{2}} + r_B^{\frac{1}{2}}} \quad \text{or} \quad Q_A = 100\left(\frac{2}{2 + 3}\right) = 40$$

$$Q_B = 100 - Q_A = 100 - 40 = 60$$

The principal has been applied by Conklin.[1] The application of the method to a complicated pipe network or to one in which relative resistances differ widely between pipes may lead to large errors, because the division of flow at each junction is controlled by the relative resistances of

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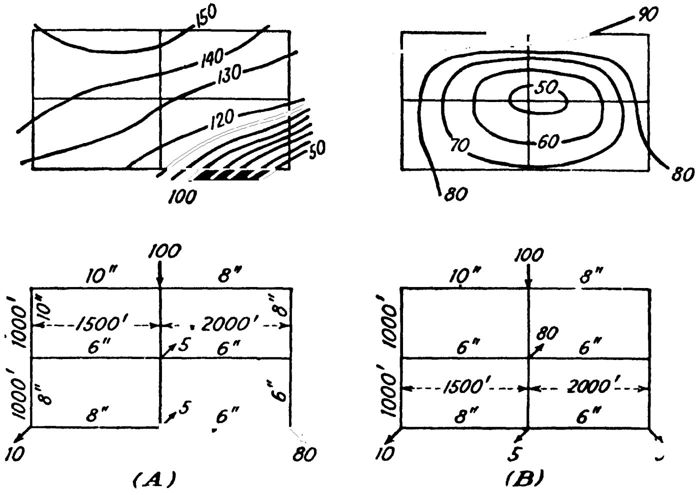
[1] CONKLIN, J. A., "Flow in Pipe Networks," *Eng. News-Record*, Sept. 11, 1941, p. 93.

all pipes *at and beyond* that junction, and not alone by the pipes at the junction.

It may be concluded that good judgment, guided possibly by a judicious application of what may be called "Conklin's method," is the proper procedure in the first assumption of flow distribution before the application of the Cross method of analysis to determine the correct distribution of flow.

390. Pressure Contours. Pressure contours may be drawn for all reasonable extremes of put in and take off in a distribution system. In general, the contours should be spaced to show a head loss of about 3 to 5 ft. per 1,000 ft. of 24-in. pipe down to about 25 ft. per 1,000 ft. of 4-in. pipe. Hydraulic deficiencies in distribution systems[1] become apparent through uneven spacing of pressure contours. Pressure contours for two conditions of put in and take off are shown in Fig. 215. The contours in Fig. 215*A* indicate that the pipes in the lower right-hand corner are too small for the assumed conditions of flow.



(A) (B)

FIG. 215. Pressure contours.

391. Service Pressures. The pressures to be maintained in a distribution system for all services other than for fire protection should lie between 30 and 100 p.s.i.; 40 to 50 p.s.i. in the water pipe in the street furnishes satisfactory service. Pressures below 30 p.s.i. are too low to supply water at a satisfactory rate to the upper stories of three- and four-story buildings. Pressures over 100 p.s.i. require heavier water pipes and greater care in construction and the maintenance of the distribution system and cause trouble in plumbing installations. The public water supply is not always delivered to the top floor of the tallest buildings in a city. It may be delivered to the fourth or fifth floor. Above that height, booster pumps must be installed in the building.

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[1] See also KINCAID, R. G., and R. E. McDONNELL, "Characteristic Deficiencies of Distribution Systems," *Jour. A.W.W.A.*, October, 1946, p. 1151.

392. Fire Pressures. Three methods are in use for furnishing fire pressure. They are as follows:

1. Full fire pressure constantly maintained in the distribution system. This method is used principally in the smaller water-supply systems, and the pressure maintained is frequently too low for satisfactory fire protection.
2. The pressure is supplied entirely from the pumping station, where, upon an alarm of fire, it is increased as required. This method of operation is used in cities of small and moderate size. As the cities increase in size, the number of fire engines in use is increased, until full dependence is placed upon them.
3. Fire pressure is secured through mobile pumping engines that draw water from a hydrant near the fire. The pressure on the pumping station is not increased under these conditions. This method of fire protection is used in the largest cities.

The proper pressure for fire protection is usually expressed as the pressure required at the fire nozzle. The National Board of Fire Underwriters recommends 75 p.s.i. where more than ten buildings exceed three stories in height; 60 p.s.i. in localities with less risk; and in thinly built-up districts, 50 p.s.i. Where fire engines are used, the board's recommendations permit a minimum pressure of 10 p.s.i. at the hydrant, under unusual conditions. A pressure of 20 p.s.i. is considered satisfactory under most conditions where fire engines are used.

393. Location of Valves, Tees, and Specials. Valves should be placed on all branches from feeder mains and between feeder pipes and hydrants. Ordinarily not more than three valves are placed at a cross or more than two valves at a tee. When there is a choice of position the valve should be placed in the smaller of two pipes joined at the cross or tee. In no case should a length of pipe greater than 1,200 ft. be left without valve control. In high-value districts 500 ft. should be the maximum. Very few systems are supplied with too many valves. In the construction of a distribution system it is customary to install only the most essential pipes in the expectation that the remaining pipes in the design will be installed as the demand increases. Provision for expansion should be made by the installation of the necessary specials, tees, crosses, elbows, bends, etc. Liberal use of specials that will permit easy connection to the distribution system is highly desirable, since it is not possible to foresee all desirable extensions at the time of design.

394. Location of Fire Hydrants. The spacing of fire hydrants is determined by the area that each hydrant serves. The area is partially governed by population, value of the district, and the method used for obtaining necessary fire pressure, *i.e.*, direct from pumping stations or portable pumpers. Table 70 shows areas per hydrant approved by the National Board of Fire Underwriters.

TABLE 70.—FIRE FLOW AND HYDRANT AREAS
(National Board of Fire Underwriters)

Population	Required fire flow for average city, gallons per minute	Average area per hydrant, square feet	
		Engine streams	Direct streams
1,000	1,000	120,000	100,000
4,000	2,000	110,000	85,000
10,000	3,000	100,000	70,000
100,000	9,000	55,000	40,000[1]
200,000	12,000	40,000	40,000

[1] 40,000 sq. ft. is the maximum allowance for direct streams in all cities having a population of 28,000 or more

The designer should study also hydrant spacing on the basis of economy. Using reasonable assumptions, the annual expense of a hydrant is less than the annual expense of two 50-ft. lengths of standard fire hose. Furthermore, the loss of pressure is usually much greater through the hose than through the pipe line serving the hydrant. There is also an advantage that accrues to the consumer through a reduction in insurance rates resulting from an approved spacing of hydrants. Hydrants should be placed at street intersections and at such intermediate points as may be required by spacing.

Some recommendations concerning hydrant location that may be modified by local conditions include the following: (1) Place hydrants more than 2 ft. and less than 6 ft. from the edge of a paved road surface, to avoid interference by traffic and to minimize the length of suction hose to mobile pumpers. (2) Place hydrants 3 ft. or more away from fixed objects, or doorways, and out of traffic lanes. (3) Set the hydrant so that the lowest outlet is 18 in. or more above ground surface and the highest operating nut is not more than 48 in. above the ground surface.

A convenient procedure in locating fire hydrants is to draw a circle with a radius of about 80 per cent of the assumed permissible length of hose, with a hydrant as the center of each circle. It may be assumed that areas not within a circle are inadequately protected. High-risk areas should be protected by at least two fire hydrants within reach of every part of the area.

395. Fire Streams. The factors of importance in a fire stream are the number of gallons per minute passing the nozzle, the height of usefulness of the stream, and the distance at which it is useful. These factors are determined by the shape, size of opening, interior smoothness, and the pressure at the base of the nozzle. A standard type of fire nozzle is illustrated in Fig. 216. The distances and heights to which various quantities of water will be thrown by fire streams are listed in Table 14. Losses of head in hose and hydrants may be determined by the methods given in Sec. 47. The effective reach of a fire stream can be computed by the Goldsmith formula[1]

$$H = \sqrt{(240P - P^2 - 1{,}900)} - 15 \qquad (9)$$

in which H is the height, in feet, for a $\frac{3}{4}$-in. nozzle, and P is the pressure on the base of the nozzle, in pounds per square inch. For larger nozzles, increase H by 1 ft. for each additional $\frac{1}{8}$ in. nozzle size for pressures below 50 p.s.i., and by 2 ft. for pressures above 50 p.s.i.

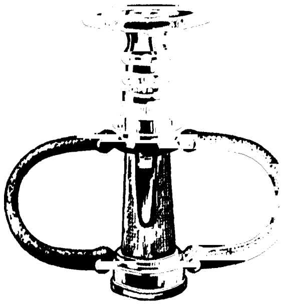

FIG. 216 Standard type of fire hose nozzle with handles. (*Courtesy of Elkhart Brass Mfg. Co.*)

The total capacity of fire streams to be provided depends upon the size of the city and the character of risk requiring protection. Empirical formulas have been recommended as stated in Sec. 54. These include:

Kuichling:

$$Q\ (\text{g.p.m.}) = 700\sqrt{P} \qquad (10)$$

National Board of Fire Underwriters:

$$Q = 1{,}020\sqrt{P}\left(1 - \frac{\sqrt{P}}{100}\right) \qquad (11)$$

in which P is the population in thousands.

A study of current practice in fire protection will show the use of streams in well-protected cities with a discharge between 175 and 250 g.p.m., with pressures on the base of the nozzle between 40 and 100 p.s.i. Goldsmith[2] recommends a minimum of 50 p.s.i. Where pressures are provided by fire engines drawing their water from the hydrant, the pressure in the distribution system need not be increased, thus removing one cause of leaks and breaks in distributing pipes.

In general it can be considered satisfactory in a residential district with no special risks, such as churches and schools, to have available for each building not less than four streams of 175 g.p.m. each with a pressure at the base of the nozzle of not less than 35 p.s.i. In business, industrial, and other districts, where risks of special value and hazard are to be protected, the capacity, number of streams, and pressures available must be increased. The recommendations of the National Board of Fire Underwriters in this regard require somewhat more generous protection than is provided by many waterworks. These recommendations are as follows:[1]

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[1] GOLDSMITH, C., "Efficient Utilization of Water for Fire Fighting," *Jour. A.W.W.A.*, December, 1939, p. 2027.

[2] *Ibid.*

> In estimating required fire flow, an allowance is made for probable loss from broken connections incident to a large fire. Including this allowance, the total fire flow which should be available is approximately as given by formula (11); but, in all cases, consideration must be given to the structural conditions as found in the city, and also to the number of companies in the fire department and the amount of outside aid that could be called upon in case of a serious fire. The ratio of the total engine capacity to the fire flow required will be approximately 2:3.
>
> In residential districts, the required fire flow depends upon the character and congestion of the buildings. Sections where buildings are small and of low height and with about one-third of the lots in a block built upon require not less than 500 g.p.m.; with larger or higher buildings, up to 1,000 g.p.m. is required; and where the district is closely built, or buildings approach the dimensions of hotels or high-value residences, 1,500 to 3,000 gal. is required, with up to 6,000 gal. in densely built sections of three-story buildings.

In grading the adequacy of the water supply, two features are considered:

> 1. The ability to deliver fire flow at any pressure down to 20 lb., except that a minimum of 10 lb. is permissible in districts having no deficiency in items . . . and having all hydrants provided with at least one steamer outlet; this is a measure of the ultimate capacity of the system to maintain supply of some sort under conflagration conditions and with the system in full operation.
>
> 2. The ability to deliver this fire flow or a part thereof at pressures permitting streams direct from hydrants; these pressures are assumed at 75 lb. or more in high-value districts, depending upon the static pressure available, the heights of buildings and the amount of engine capacity in service; 60 lb. where not more than ten buildings exceed three stories and in closely built residential districts, and 50 lb. in village mercantile districts where buildings do not exceed two stories and in thinly built residential districts. . . .

High-voltage electric transmission lines in a fire area may be a source of danger to fireman because of the possiblity that an electric current may follow the fire stream and cause loss of control of the nozzle or direct injury from electric shock. Tests by Sprague and Harding[2] indicate that the relative danger is dependent upon the sensitivity of the individual, the voltage of the transmission line, the distance of the nozzle from the line, and the resistivity of the water which may vary from 500 ohms per milliliter for mineralized well water to 6,000 for filtered river water. Allowing a generous factor of safety, minimum distances from nozzle to transmission line for a 3,000-ohm water should be 3 ft. for a 440-volt line and 33 ft. for a 13,200-volt line. The stream should not be allowed to strike the line if the voltage is higher than 13,200.

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[1] "Standard Schedule for Grading . . . Fire Defenses," National Board of Fire Underwriters, adopted Dec. 14, 1916, edition of 1942, p. 14.

[2] "Electrical Conductivity of Fire Streams," *Purdue Univ., Eng. Bull.* 53, 1936.

396. Studies of Existing Systems. Studies[1] to determine the capacity of an existing distribution system are made by direct observation of flows and pressures. Direct observations may reveal closed valves, air pockets, errors in records of pipe connections, and other conditions.

The methods used involve the construction of pressure contours during normal and fire demand. Points on the pressure contours are determined by observations of pressure at selected points on the distribution system. Because of fluctuations in pressure, observations are best made by recording gages. In the study of fire demand groups of four to six hydrants are selected along the larger mains. A central hydrant in each group, preferably taking water directly from the large main, is used to include the static pressure in the group, before and during the test, when water is drawn at measured rates from all other hydrants in the group.

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[1] See also BOOTH, G. W., "Practical Studies of Distribution Systems," *Jour. A.W.W.A.*, September, 1944, p 957 and KINCAID, R. G., "Analyzing Distribution Systems," *Water Works Eng.*, Oct. 18, 1944.

CHAPTER XXIII

CONSTRUCTION AND MAINTENANCE OF DISTRIBUTION SYSTEMS

397. Safe Practice. In the construction and maintenance of a waterworks-distribution system the "Manual of Safe Practice in Water Distribution,"[1] will be found to be a valuable guide. It contains standards and specifications applicable in design, construction, and maintenance.

398. Work Involved in Construction. The laying of water pipe involves transportation and handling of the pipe and materials, ditching and excavation, backfilling, rights of way, and other problems common to construction work. Only conditions peculiar to the laying of water mains are considered here, because the space required for a proper consideration of all construction methods is not available.

When pipes, specials, valves, and other appurtenances to a waterworks distribution system are delivered on the site of the work, they should be inspected for defect, and all rejected parts should be immediately removed from the work. The following extracts are taken from specifications covering the laying, testing, and acceptance of cast-iron water mains:[2]

> *Pipe Laying.* Each pipe shall be laid upon a firm bed and, except on curves, all pipes shall be laid in a straight line. Excavation must be made under the bell of each pipe, so that the entire length of the pipe, except the bell, may be supported on the bottom of the trench. All pipes, specials, valves, and appurtenances shall be cleaned by brushing and by thoroughly scraping out all dirt or other foreign matter before laying.
>
> The ends of pipe shall closely abut upon each other, and the spigot shall be concentrically placed in the hubs or bells, so as to admit a uniform thickness of gasket and lead. The calking shall consist of the specified quantity of well-rammed untarred rope yarn or jute above which shall be poured a sufficient quantity of melted lead to stand flush with the outside of the bell of the pipe when the lead is compacted. The lead shall be compacted with a calking hammer and thoroughly driven around the entire circumference of the pipe.
>
> The contractor shall put caps or plugs on all unconnected ends of pipe and on all unconnected crosses, tees, and branches or wyes. Pipes laid around

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[1] Published in *Jour A.W.W.A.*, June, 1942, p. 917.

[2] See also "Tentative Standard Specifications for Laying Cast Iron Pipe," *Jour. A.W.W.A.*, Vol. 30, p. 215, 1938.

> curves shall be properly blocked with brick or concrete to take the reaction, and all plugs and blanked openings shall be secured by masses of concrete.
>
> The work of laying the pipes and special castings and of setting of valves, hydrants, and appurtenances shall be of such character as to leave all the pipes and connections watertight. To assure these conditions, the contractor shall subject the pipe and all appurtenances to a proof of water pressure of not less than 110 p.s.i.
>
> *Testing Pipe Joints in Trenches.* The tests shall be made between valves and, as far as practicable, in sections of approximately 1,000 ft. in length and within 12 working days of the completion of such sections of mains. The leakage from the mains and connections for each section tested, while the pressure is at 110 p.s.i., shall not be greater than at the rate of 1,800 gal. per 24 hr. per mile of pipe, for pipe 14 in. in diameter or less. To determine the rate of leakage, the contractor shall furnish a suitable pump, pressure gage, and water meter, or other appliance, for measuring the amount of water pumped. The pressure shall be raised to 110 lb., and it shall be so maintained for a period of not less than 20 min. If the leakage be at a greater rate than that specified, the contractor shall reexcavate the trench where necessary and shall recalk the joints and replace defective work until the leakage shall be reduced to the allowable amount.
>
> Where it is impracticable to test between valves, or near connections to existing mains, the contractor shall . . . temporarily place caps or plugs on the mains and test the section of the main so closed.

Pipes should usually be laid with the bell end upgrade for greater ease in making joints. In general, water and sewer pipes should not be placed in the same trench but, where otherwise unavoidable, the water pipe should be laid higher than the sewer and to one side of it.

399. Raising and Lowering Water Mains. Maintenance work may involve the changing of the elevation of water pipes owing to regrading and paving of streets. Water pipes have been raised or lowered under pressure, without seriously affecting service, by placing blocks and jacks under the pipes and changing direction a little at a time. There is appreciable flexibility in lead joints to permit movement of the pipe.[1] Even sulphur and cement joints allow some deflection. Permissible deflections depend on the type of joint compound, pipe diameter, and internal pressure.

400. Disinfection of Water Mains.[2] Water mains must be disinfected before being put into use and, sometimes, during use.[3] Before disinfecting, the pipe should be flushed, drained, and refilled. In refilling, care must be taken to avoid entraining or entrapping air in the pipe, preventing contact between the pipe and the disinfectant. The pipe is subjected to any one of the following three treatments, stated in order of preference: (1) A chlorine gas-water mixture is applied at the beginning of the pipeline extension, or any valved section of it, at such a rate that with a slow flow of water into the pipe the chlorine dose shall not be less than 50 p.p.m. The treated water is to remain in the pipe at least 3 hr. and preferably longer. At the end of the period the chlorine residual is not to be less than 5 p.p.m. Following chlorination the pipe is thoroughly flushed, tested, and, if necessary, rechlorinated until a satisfactory bacteriological test is obtained. (2) A solution of calcium hypochlorite or such commercial products as HTH, Perchloron, and Maxochlor or chlorinated lime may be used in the place of chlorine gas. After the pipe has been subjected to a preliminary flushing, a 5 per cent solution of any of the afore-mentioned powders should be injected into the pipe in a manner similar to that required for the addition of the liquid chlorine. (3) Dry calcium hypochlorite or chlorinated lime may be used in such a manner that there shall be 1 lb. of powder, containing 70 per cent available chlorine, for every 1,680 gal. of water in the pipe, equivalent to 50 p.p.m. of available chlorine. The dry powder is to be shaken into the suction well of a pump or into a standpipe feeding the pipe line, or it may be shaken into each length of pipe as it is laid. After the materials are introduced, water is admitted very slowly so as to avoid washing the powder to the end of the line. After standing for a few hours the pipe should be flushed until no odor of chlorine can be detected in the water or until a check can be made for residual chlorine.

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[1] See also BUSHWAY, W. B., "How To Raise and Lower Mains under Pressure," *Am. City*, September, 1914, p 69; and ROBIE, K. W., "Deflection and Leakage Tests," *Am. City*, September, 1914, p. 71.

[2] See also HELLER, A. N., "The Disinfection of Water Mains," *Jour. A.W.W.A.*, December, 1946, p. 1335.

[3] Tentative standard specifications of the A.W.W.A., *Jour. A.W.W.A.*, Vol. 30, p. 235, 1938, require that all new pipe lines shall be chlorinated and provide various methods therefor.

Jute packing and leathers in pumps, valves, and hydrants are difficult to disinfect and offer a focal point for the growth of bacteria. Methods used in the sterilization of jute and hemp, before it is used in a joint, include[1] (1) soaking in chlorine solutions of 50, 100, or 500 p.p.m. of available chlorine, for 5 min. to 2 hr.; (2) autoclaving with live steam under pressure for 1 to 30 min.; (3) boiling for 1 to 30 min; and (4) baking in dry heat. Autoclaving with live steam under pressure has proved most effective. Klerol (ortho-mercuro-phenol) is reported by Calvert[2] to be a satisfactory disinfecting agent.

401. Work Involved in Maintenance. Maintenance of a distribution system involves (1) keeping records; (2) finding, cleaning, and flushing pipes; (3) installing, cleaning, and thawing service pipes; (4) preventing waste; (5) finding and repairing leaks; (6) thawing frozen pipes, hydrants, and meters; and (7) inspecting, caring for, and repairing hydrants, valves, meters, and other appurtenances.

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[1] See also "Methods and Efficiency of Disinfecting Jute Packing," *Public Works*, June, 1943, p. 18.

[2] CALVERT, C. K., "Disinfection of New Mains," *Jour. A.W.W.A.*, January, 1945, p. 46.

402. Records. Knowledge of the location of every part of a waterworks distribution system is essential to proper operation and maintenance.[1] Experience shows that poor records result in "lost" valves, closed or partly closed valves, valves duplicated, mains dead-ended near each other, and other costly and detrimental effects of carelessness and forgetfulness. Good records sometimes discourage, and sometimes help to win, damage suits, aid in establishing water rights, and help to maintain good public relations.

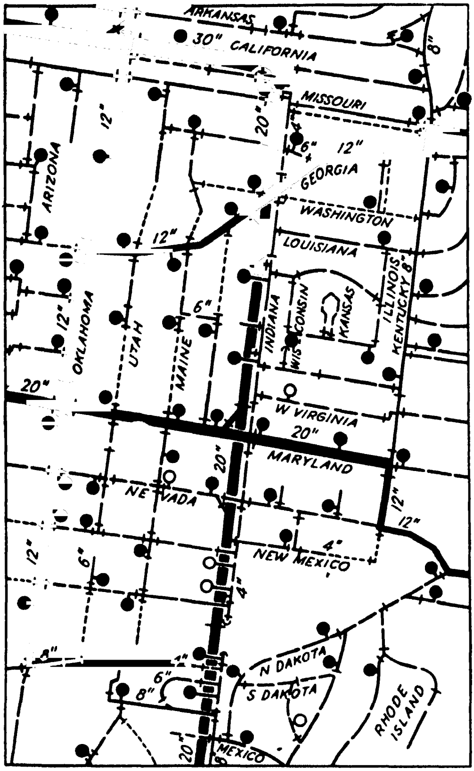

FIG. 217.- Portion of comprehensive map. (*From "Recommended Practice for Distribution System Records," Jour. A W W.A., February, 1940, p. 188.*)

Maps of the distribution system are the most important record to be

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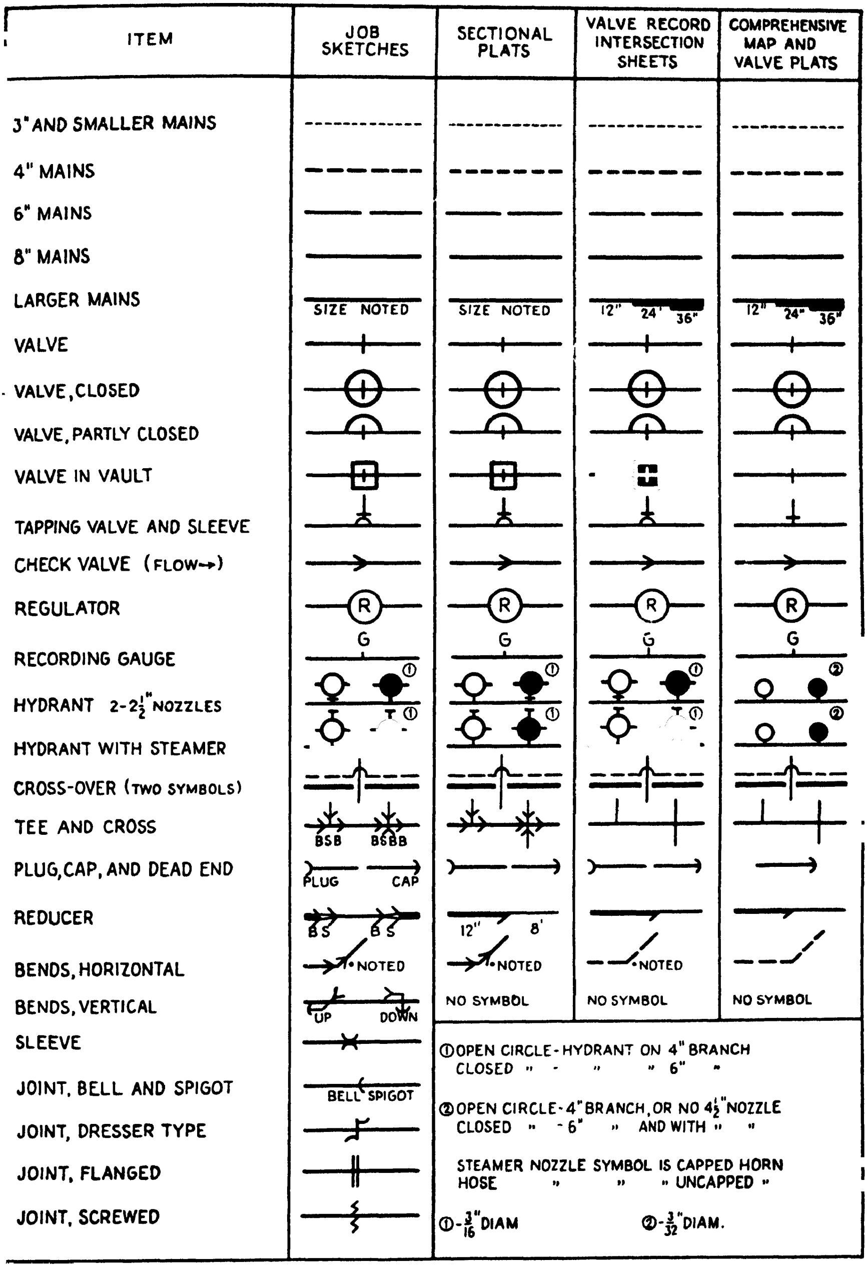
[1] See also Committee Report, "Recommended Practice for Distribution System Records," *Jour. A.W.W.A.*, February, 1940 p. 181.



①OPEN CIRCLE-HYDRANT ON 4" BRANCH
CLOSED " - " " 6" "

②OPEN CIRCLE-4" BRANCH, OR NO 4½" NOZZLE
CLOSED " -6" " AND WITH " "

STEAMER NOZZLE SYMBOL IS CAPPED HORN
HOSE " " " UNCAPPED "

①-$\frac{3}{16}$" DIAM ②-$\frac{3}{32}$" DIAM.

Fig. 218.—Standard symbols for water-distribution systems. (*From "Recommended Practice for Distribution System Records," Jour. A.W.W.A., February,* 1940, *p.* 216.)

maintained. Three types of maps are desired: (1) a comprehensive map of the entire system; (2) a larger scale map showing details; and (3) plats and cards providing information on valves, hydrants, and other appurtenances.

A portion of a comprehensive map is shown in Fig. 217, and symbols to be used are shown in Fig. 218. Information to be included on the comprehensive map is listed in Table 71. The sectional plats provide a detailed record of distribution-system structures. Information to be shown on such plats is listed in Table 72.

TABLE 71.—INFORMATION CONCERNING THE COMPREHENSIVE MAP

Material: Original—heavy tracing cloth
Prints—black line preferred
Scale: Preferred 500 ft. per in.
Maximum—1,000 ft. per in.

Items to Be Shown on the Map

1. Street names
2. Sizes of mains
3. Fire hydrants
4. Valves
5. Orientation arrow
6. Scale
7. Date last corrected
8. Complete title

TABLE 72.—INFORMATION TO BE SHOWN ON SECTIONAL PLAT

Material: Original—heavy tracing cloth
Prints—black line preferred
Size: 20 by 25 in. plus 2- to 3-in. margins
Scale: 50 ft. per in. in congested areas
100 ft. per in. in residential areas
200 ft. per in. in rural areas
Index: Coordinates; horizontal spaces lettered, vertical spaces numbered; or horizontal spaces even numbers and vertical spaces odd numbers.

Items to Be Shown on Plats

1. Plat designation or number
2. Adjacent plat numbers
3. Street names and widths
4. Mains and sizes
5. Materials of mains
6. Years mains were installed
7. Work orders of main installations
8. Distances from property lines
9. Fire hydrants, numbers and classifications
10. Valves and numbers
11. Valve sheet designation shown in margin
12. Intersection numbers (if valve intersection plats are used)
13. Block numbers
14. Lot numbers
15. House numbers
16. Water account numbers
17. Measurements to service lines
18. Sizes of taps
19. Sizes and materials of service lines
20. Distances, main to stop box
21. Distances, stop box to property line
22. Distances to angle points
23. Distances to fittings
24. Dead ends and measurements
25. Date last corrected
26. Orientation of north arrow
27. Scale

A valve record and a hydrant record should be kept because of the importance of quickly obtaining information concerning them. Among the things to be included on the valve record sheet are the size and make of the valve, direction to open, turns to operate, its number, street or streets on which located, and distance and direction to near-by reference points. Card records may supplement maps and charts, the cards giving information not shown on the map or chart.

A well-maintained waterworks will include the following in a monthly record or report: (1) the length and sizes of pipes laid and retired during the month and the total mileage of each size laid to date; (2) the number of each size of valves; (3) the number, size, and types of fire hydrants; and (4) the number, size, and lengths of services. In keeping records of work done and its cost the "work-order" system should be followed. In this system a card record shows location and nature of work; labor, materials, and equipment used, reclaimed, or retired; and the time involved.

403. Scraping Water Mains.[1] Water pipes require cleaning principally to remove soft deposits precipitated from the water and to remove tuberculation and incrustation from the pipe walls Other materials are sometimes removed in cleaning, such as lost tools, bolts, and valve parts. The economy of cleaning has been frequently demonstrated through increased pipe capacity.[2] Pipes are cleaned by pushing or pulling a scraping machine through them, such as the machine shown in Fig. 219.

A common procedure is to insert the scraping machine into the empty pipe through a *Y* or *T* and to push the machine through the pipe by water pressure behind it. A cable may be attached to the scraper to permit its recovery or to indicate its location if it sticks. The water pressure is used to drive the turbine blades if they are included on the scraper. Water passing through, or around, the scraper flushes the scrapings and soft deposits ahead of the scraper. The deposits may be removed through a hydrant at a lower elevation on the pipe, or through the fitting through which the scraper is removed from the pipe. A successful turbine scraping machine may restore the pipe capacity, but it may also injure the pipe lining; therefore once scraping has been started it must be repeated more or less frequently. Scrapers travel about 1 m.p.h. and require a pressure of about 65 p.s.i. when cleaning a 12-in. pipe. Some mechanical scrapers that fit the pipe can be dragged through the pipe to clean it.

404. Cleaning with Acids. Acids may be used successfully, particularly in small pipes, for the removal of deposits of calcium carbonate provided an inhibitor is combined with the acid. An acid inhibitor is a substance that inhibits the release of acid when an acid comes into contact with a metal. The film of hydrogen held by the inhibitor protects the metal against attack by the acid. The inhibitor does not inhibit the action of the acid on the calcium deposit. Inhibitors that have been used successfully include bran, flour, glue, and such other organic substances as aniline, pyridine, and quinoline. Inhibitors containing arsenic should not be used.[1] Before any material is selected as an inhibitor, its effectiveness as such should be tested for its rate of hydrogen evolution from a standard metallic sample, under the same temperature, acid concentration, and other conditions in which it is to be used in the field. Mick[2] reports that a 0.7 per cent hydrochloric acid solution and 2 per cent

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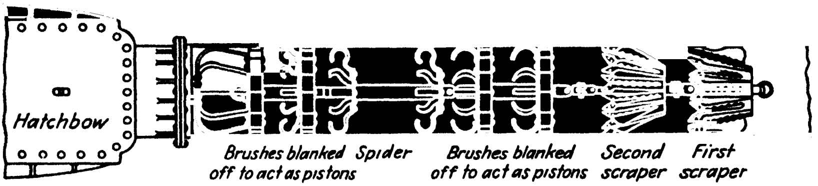
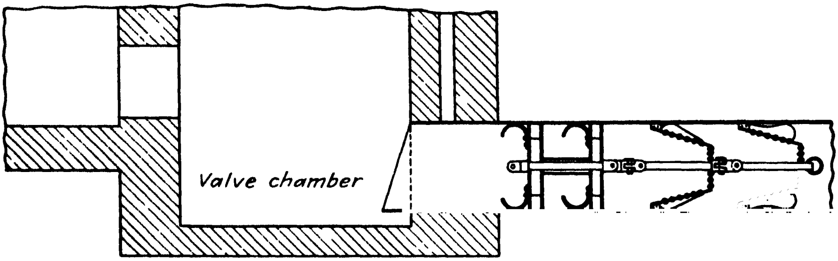
[1] See also Bailey, J. F., "The Scraping of Large Water Mains," *Water and Water Eng.*, August, 1943, p. 317.

[2] See also Brown, J. R., "Is Water Main Cleaning Good Economy," *Water Works & Sewerage*, February, 1944, p. 72.


Hatchbow
Brushes blanked off to act as pistons
Spider
Brushes blanked off to act as pistons
Second scraper
First scraper

DIAGRAMMATIC SKETCH OF COMPLETE SCRAPING ASSEMBLY



DIAGRAMMATIC SKETCH OF SHORT SCRAPING ASSEMBLY

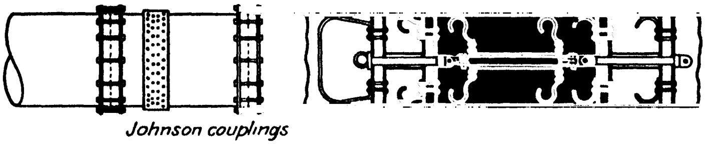


Fig. 219. Water pipe scraper. (*From J. F. Bailey, Water and Water Eng., August*, 1943, *p.* 318.)

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[1] See also Hollett, A. R., "The Use of Inhibitors in Acid Solutions Employed to Clean Water Pipes," *Jour. A.W.W.A.*, May, 1937, p. 663; and Wordell, V. A., "Use of Inhibitors in Preventing Corrosion by Acids," *Chem. and Min. Rev.*, Vol. 27, p. 286, 1935.

[2] Mick, K. L., "Cleaning Encrusted Pipe Lines and Vacuum Filters with Inhibited Muriatic Acid," *Water & Sewage Works*, April, 1946, p. R-241.

aniline oil dosage will cut corrosiveness of acid by 44 per cent, and a 2 per cent aniline oil dosage in a 5 per cent acid solution will cut corrosion by 97 per cent.

405. Emptying Water Mains. Water mains must often be emptied before cleaning. This may be done by opening a blowoff valve or other outlet in the lower end of the pipe and opening a hydrant, service connection, or other opening at the upper end to admit air. Under favorable circumstances some of the water may be drained from the main through a hydrant, only the remainder below the hydrant level draining into the excavation from which the water must be pumped. In some cases compressed air, admitted at a high point, has been used to expel the water from the main through a hydrant, thus expediting the emptying of the main and avoiding pumping.

406. Flushing Dead Ends. Because of the accumulation of rust, organic matter, and other material in the quiet water in dead ends, undesirable odors, tastes, and color, occasionally accompanied by putrefaction, sometimes occur. Routine inspection and thorough flushing may be the remedy. Improper or inadequate flushing can make conditions worse by stirring up sediment without removing it. Flushing is usually accomplished by opening a hydrant wide and allowing water to run until clear. In a few cases compressed air has been forced into the main to increase velocity and agitation.

407. Air Locks. Loss of capacity of water pipes, either in a long pipe line or in a network, may result through the accumulation of air in a high point in the line or network. Such air locks cannot be forced. They may be relieved by the installation of an air valve or prevented by lowering the pipe.

408. Cleaning Service Pipes. Service pipes clog more easily than mains owing to tuberculation and sediment. Service pipes must be cleaned by dragging or pushing a scraper or by blowing compressed air through them, since they are too small to permit the use of a turbine scraper. Tools such as cutting knives, brushes, drags, and scrapers that can be attached to flexible rods and twisted by power are available.[1] Sometimes the pushing of a soluble object such as toilet paper through the service pipe is helpful. Trentlage[2] reports beneficial results in 70 per cent of the attempts, and bursting of pipes in only 10 per cent, when suddenly releasing a mixture of air and water at a pressure of 1,400 p.s.i. into a clogged service pipe.

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[1] See also CARLTON, L. G., "Installation and Care of Services," *Water Works Eng.*, Sept. 11, 1940, p. 1150.

[2] TRENTLAGE, LAV., "Service Shooting," *Water Works & Sewerage*, June, 1943, p. R-113.

409. Cleaning and Disinfecting Reservoirs. The cleaning of reservoirs is a routine maintenance duty. The frequency of cleaning and the methods used depend on local conditions and ingenuity. Methods used include draining, hosing, scrubbing, scraping, and refilling. Disinfecting may be accomplished by draining and refilling at the same time that a dose of hypochlorite is applied to give a concentration of free chlorine of about 1 p.p.m. to the water.

410. Waste Surveys and Control. A large amount of water passing through a pumping station may be unaccounted for. Cities with all services metered are sometimes unable to account for as much as 20 to 30 per cent of their pumpage. However, the expression of unaccounted-for water as a percentage of the pumpage may be misleading since the metering of one large consumer may decrease the percentage while the quantity of water wasted will remain the same.

Unaccounted-for water is not necessarily all leakage. Some of it may represent low flows passing through meters too large for the service, and some may represent stolen water. Leakage and waste surveys have commonly reduced both total consumption and unaccounted-for water. Leaks are undesirable both because they waste water and because they dangerously undermine pavements and structures. Abandoned service pipes are a prolific cause of leaks.

Waste surveys are conducted by isolating a section of the water distributing system by closing the valves controlling the flow of water thereto so that all the water to the isolated district passes through a single pipe. The flow through this pipe is measured, and if it is found to be greater than normal, the "isolated" district is divided into smaller districts and the source of abnormal consumption located more closely. The rate of water flow may be measured by inserting a pitometer into the pipe.[1] As this may necessitate uncovering the pipe by excavation and boring a hole for the insertion of the pitometer, the following method is sometimes used. All the valves in a section of the waterworks distribution system are closed, and water is fed to the district through one or more fire hoses, equipped with meters, which are connected to hydrants on either side of the closed valves.

When excessive consumption is located in a particular district, house-to-house inspection of plumbing may be conducted to locate the waste. Theft of water can sometimes be detected by such surveys.

411. Illegal Use of Water. The illegal use of water is usually not a serious economic matter, but it should be guarded against as a matter of principle. Illegal methods include by-passing around the meter, called a

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[1] See also COLE, E. S., "Present-day Scope of Pitometer Surveys," *Jour A W. W.A.*, February, 1945, p. 181.

"jumper"; inducing low, unrecorded flows through the meter, called "dripping"; in flat-rate services, supplying a neighbor through a hose or turning on water without notifying the water company; drawing water from hydrants without permission, most commonly done by builders; tampering with or reversing the recording mechanism on the meter; wrecking the meter dials; and using hidden connections beyond the meter. Alert and observing meter readers and other waterworks employees, encouraged by rewards for the discovery of illicit use of water, are the best cure for such abuses.

412. Finding Pipes. Where records are inadequate, underground and subaqueous pipes may become lost. They must usually be located before leaks in them can be discovered. Pipe locators are of two types: one works with the pipe as a part of a closed electric circuit; in the other an aerial picks up the magnetic field induced in the pipe by a radio transmitter or by previous excitation. In the former, after the electric circuit has been completed by electric contacts with two exposed points on the pipe line, the magnitude of sound in earphones will increase as the pipe is approached and will fall silent when the observer is directly over the pipe. In the latter an electric dip needle points to the buried pipe.[1]

413. Finding Leaks.[2] Methods used for the location of leaks in underground pipes and in subaqueous pipes include (1) direct observation, (2) the sounding rod, (3) the hydraulic grade line, (4) water hammer, (5) blowing air into the pipe, (6) putting coloring matter or salt into the water, (7) measuring the volume of water required to fill the pipe, and (8) listening either directly for the sound or with the aid of a sonoscope or electronic sound amplification. Each method is best suited to particular conditions. None is universally applicable or infallible.

Direct observation can scarcely be called a method. It would seem to require no special knowledge to say "There is a leak," when water is seen gushing up in a city street. However, some skill may be needed to detect a leak where there is a luxurious green spot in a lawn, a soft spot in the ground, a melted patch of snow or ice, or where a spot of frost or dew disappears quickly in the early morning. Any one of these clues may indicate the location of a leak, as may the sudden or otherwise inexplicable increase in the flow in a sewer[3] or the appearance of a "spring" where there should be no spring. Water does not always appear close to the spot from which it left the pipe. After the general location of the leak has been determined, the sounding rod may be required to find it exactly.

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[1] See also MILLER, BASIL, "Miller Pipe Locating Magnet," *Water Works & Sewerage*, September, 1943, p. 335; and FISHER, C. R., "Electronic Pipe Finding and Leak Locating," *Jour. A.W.W.A.*, December, 1946, p. 1330.

[2] See also BABBITT, H. E., "The Location of Leaks in Underground Pipe," *Jour. A.W.W.A.*, Vol 7, p 589, 1920.

[3] See also GROSS, D. O., "Water Waste Detection by Observing Sewer Flows," *Water Works & Sewerage*, June, 1944, p. R-172.

This instrument is a sharp-pointed metal rod that is thrust into the ground and pulled up for inspection. If moist or muddy, the line of the leak is being followed.

If a pipe leaks so badly that it will empty itself in a short time after it has been shut off, the leak can be found by allowing the pipe to drain. Then the volume of water required to fill the pipe quickly should be measured and this figure divided by the cross-sectional area of the pipe. The quotient will be the distance to the leak.

Leaks in water pipes usually make some sound unless the leak is submerged. Small leaks are likely to make more noise than large ones. The leak may be heard directly; it may be heard by placing the ear on the service pipe in the basement; or a sounding rod may be pushed down until it comes in contact with the pipe and the ear placed against the rod. Where the unaided ear cannot detect the sound, one of the many simple instruments for magnifying the sound may make the sound audible. These include the aquaphone, the detectaphone, and the sonoscope. The first named is similar to a telephone receiver, without electrical connections. One end of the receiver is placed in contact with the sounding rod or pipe, thus making the sound audible. It is followed until the leak is located. Another device works like a stethoscope.

A leak may be detected by following the pipe in the direction of diminishing pressure, *i.e.*, following the hydraulic grade line. If two points can be found on the hydraulic grade line on each side of a leak in a long pipe line and the hydraulic grade lines are passed through these two points on a profile, they will intersect over the location of the leak.

In the application of the phenomenon of water hammer to finding a leak, favorable conditions require a long pipe line without important branches. A valve on the line is opened until the rate of flow becomes constant, and then the valve is *suddenly* closed. The instant of time that the valve is closed and the pressure in the pipe immediately thereafter are recorded. The pressure wave caused by the water hammer will travel along the pipe to the leak, where a part of the pressure wave will be dissipated. The wave of diminished pressure will then travel back to the pressure gage at the valve, and a drop in pressure will be observed. Since the velocity of travel of the pressure wave is as shown by formula (10) in Sec. 192, and the wave has traveled twice the distance between the valve and the leak, the distance to the leak, in feet, is equal to

$$D = \frac{TV}{2} \qquad (1)$$

in which T is time, in seconds, for the pressure wave to return to the valve and V is the velocity of travel of the pressure wave.

The blowing of air or the placing of coloring matter in a pipe is useful only in submerged pipe. The air bubbles or the coloring matter will appear over the leak. Fluorscein and the slightly cheaper dye uranine, both coal-tar dyes, are the most commonly used dyes. They are detectable in a concentration of less than 1 p.p.m. by the naked eye and up to 1 part in 10 billion with the aid of a fluoroscope. The sample of water observed must be alkaline since acidity bleaches the color which is restored by adding alkalinity. Such dyes cannot be used in potable water since their appearance is undesirable. A slightly increased concentration of salt, NaCl, may be used in the detection of a leak by showing, by chemical analysis, an increased chloride content in the water suspected of escaping from a pipe into which a dose of salt has been placed. The salt concentration is insufficiently increased to be detected by taste.

414. Electrical Sound Magnification. The sound emitted by a leak may be picked up by a delicate crystal and magnified up to 10,000 times by vacuum tubes. It is possible to tune the receiver to the pitch of the sound of the leak, excluding interfering sounds. In the M-Scope[1] leak locator, a very sensitive instrument, the sound of the leak is followed directly. The effectiveness of the instrument is greatly increased by making direct contact at points along the pipe and following along the pipe until the sound can be heard without contact with the pipe. Sound will travel faster and a hundred times as far through a pipe as through porous ground.

415. Permissible Leakage. Some leakage is to be expected from water pipes laid with the best of care and under favorable conditions. The "Water Works Manual" of the American Water Works Association states: "The leakage from all types of supply mains is comparatively small, although some leakage is unavoidable. A commonly prescribed upper limit for leakage is an amount variously expressed, equivalent to 250 gal. per 24 hr. per inch of diameter of pipe per mile. Thus, a 60-in. pipe would be accepted if its leakage were 15,000 gal. per mile per day."

416. Pressure Tests. The stoppage of the pipes in a distribution system and increases in the rates of consumption, of waste, or of leaks may cause falling off of pressure in some part of the system. The pressure throughout the system should be determined by attaching pressure gages to hydrants and in other desirable places. An abnormally low pressure will indicate trouble between the point of low pressure and the nearest point of normal pressure. Further observations of pressure will locate the seat of difficulty more exactly. The cause of the loss of pressure can then be determined. If a leak or high consumption is the cause, there will be a high rate of flow at the point of normal pressure. If the pipe is partly stopped, the rate of flow through it will be low.

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[1] Fisher, C. R., "Electronic Pipe Finding and Leak Locating," *Jour. A.W.W.A.*, December, 1946, p. 1330.

The rate of flow through a distribution system is sometimes so low as to render pressure observations made under conditions of ordinary flow of no value in determining whether or not the pipe is clogged. The stoppage has no effect on ordinary flows, but it may cause serious losses of pressure when fire is being fought. Observations of pressure must, therefore, be made at hydrants when water is flowing under fire-fighting conditions. Abnormally low pressures will indicate stoppage or inadequate capacity of the distribution system.

417. Freezing of Water in Pipes. The freezing of water in pipes is due to the exposure of the water to a temperature below 32°F. Time, temperature, volume of water, velocity of flow, and environment are factors affecting the rate of freezing. When water is motionless, the rate of freezing can be expressed as

$$t_1 = 2.3\frac{q}{k}\log\frac{T_2}{T_1} \tag{2}$$

where t_1 = time, in hours, for change in temperature
T_1 = original difference in temperature between water and air
T_2 = final difference in temperature between water and air
q = amount (volume, not rate of flow) of water, in kilograms, in 1 m. of pipe
k = loss of heat, in kilogram-calories per hour, for a meter length of pipe and for the type of insulation considered for 1°C. difference of temperature

Values of k[1] vary from about 0.72 for a poorly insulated pipe to 0.08 for good insulation. Since water in motion is slow to freeze, consumers frequently waste water to prevent freezing; in a few instances water companies have encouraged this practice. It is to be noted that this formula does not give the time required to cause freezing but only the time to change the temperature of the water. Freezing begins when the temperature reaches 0°C. The time t_2, in hours, required to freeze water starting at an initial temperature of 0°C. is

$$t_2 = \frac{80q}{kT} \tag{3}$$

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[1] See Bleich, S. D., "Protection of Water Services in Exposed Locations," *Jour. A.W.W.A.*, Vol. 18, p 564, 1927.

where T is the temperature of the outside air expressed in degrees below 0°C., and q and k are the same as in formula (2).

When water freezes, it expands with a force that cannot be resisted by any pipe manufactured. It expands about one-twelfth of its volume in freezing: *i.e.*, 12 cu. ft. of water will become 13 cu. ft. of ice. The pressure and the increase of volume usually cause the bursting of the pipe. The freezing of service pipes is more common than the freezing of water mains, because of the smaller volume of water to be cooled in the service pipe, the longer time when there is no flow in it, and the greater exposure to which service pipes are often subjected.

Water pipes should be laid below the level of frost penetration. The rate of frost penetration is a function of the duration and the intensity of the air temperature below freezing. Frost will penetrate[1] open gravelly soil more quickly but less deeply than tight clay soil. Pavements are generally poor insulators; snow is a good insulator.

418. Thawing Frozen Pipes. The thawing of frozen pipes is accomplished by digging down to the pipe and building a fire in the trench over it, by the use of gasoline torches, by wrapping the pipe in rags and pouring on hot water, by blowing steam on or into the pipe, or by the use of electricity. When electric current is not available, steam under 3 to 4 p.s.i. pressure can be blown through a small pipe inserted in the frozen one. All but the use of electricity are generally messy, and the use of open flames within a building is dangerous.

Thawing by electricity[2] is quick, relatively inexpensive, and can be safe. The factors involved can be formulated as

$$H = I^2Rt \tag{4}$$

where H = heat, joules
I = current, amp.
R = resistance, ohms
t = time current is flowing, sec.

The amount that the temperature is raised depends on the current I and the resistance R of the pipe per unit of length. A current of 100 amp. will heat 1,000 ft. of 1-in. pipe just as rapidly as it will heat 10 ft., but it will take one hundred times the voltage to do so. Some data concerning currents, voltages, time, and other conditions in thawing frozen pipes are listed in Table 73.

It is to be noted that cast iron heats most readily and copper the least readily. Care must be taken to use the lowest amount of current suitable for the pipe material and to avoid melting lead in pipe joints. Sulphur joints do not prevent the electrical heating of pipes, although it may require higher voltage than otherwise.

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[1] See also "Frost Penetration Story Revealed by Group of Buried Thermometers," *Water Works Eng.*, May 27, 1936, p. 739.

[2] AMSBARY, F. C., JR., "Electrical Thawing," *Water and Sewage Works*, April, 1946, p. R-40.

TABLE 73.—CONDITIONS AFFECTING THE THAWING OF FROZEN PIPES BY ELECTRICITY

Diam. of pipe, in.	Pipe material	Resistance, ohms	Length, ft.	Approx.[1] volts	Amp.	Time, min.	Size of leading wire to use	
							Amp.	Size, B. & S.
¾	Wrought iron	82.80	600	60	250	5	200	0
1	Wrought iron	.....	600	60	300	10	225	00
1½	Wrought iron	....	600	60	350	10	275	000
2	Wrought iron	.....	500	55	400	15	325	0000
3	Wrought iron	......	400	50	450	20		
4	Cast iron	684.	400	50	500	60		
6	Cast iron	.	400	50	600	120		
8	Cast iron	. . .	300	40	600	240		
¾	Copper	9.35	400	40	500	30		
1	Copper	.	400	40	600	60		
1¼	Copper		300	35	600	60		
	Steel	63						
	Lead	123						

Relative Resistance of Pipe Metals, Expressed in Ohms per Mil Foot of a Piece of Metal 0.001 In. in Diameter and 1 Ft. Long

Metal.....	Copper	Steel	Wrought iron	Lead	Cast iron
Resistance	9.35	63.0	82.8	123.0	684

[1] The lowest possible voltage should be used.

In thawing frozen pipes electrically the procedure is to include the length of frozen pipe in an electrical circuit in which the current and voltage can be controlled. Ideal welding sets are available for this purpose. In thawing a service pipe all possible paths for the current to enter the house must be broken. One line from the transformer is connected to the service pipe where it enters the cellar. The other line may be connected to a near-by hydrant, to an exposed point on the service pipe beyond the frozen spot, or to a neighboring service likewise disconnected from its house. The cost of electricity used is insignificant, amounting to 1 to 10 cents per service thawed.

419. Care of Hydrants. The principal dangers to fire hydrants are breakage by traffic; injury through ignorance, carelessness, or abuse; and freezing. Traffic injuries may be avoided by setting the hydrant in a safe location; malicious abuse may be prevented by police- and fire-department protection; and the effects of other injuries can be minimized by frequent inspection, test, and quick repairs. Cooperation with the fire department is usually effective.

Frozen hydrants are a serious menace. They can be protected from freezing by covering them with a quickly removable box. This has the advantage also of leaving the top of the hydrant free from snow and ice. The use of antifreeze in the hydrant is undesirable because of the possibility of polluting the water supply. Hydrants may freeze owing to the freezing or clogging of the drain pipe which permits water to stand in the barrel. The fact that a hydrant is frozen can be detected by dropping a small weight, tied to the end of a string, into the barrel through the nozzle. The difference in the sound when the weight hits water, ice, or the valve seat is easily recognized.

Frozen hydrants may be thawed by the use of salt, NaCl, or calcium chloride, $CaCl_2$, and by methods applicable to the thawing of frozen pipes. The action of salt and of calcium chloride is usually slow. Heat from an insulated wire wound around the exposed portion of the hydrant, with adequate current sent through the wire, will be transmitted down through the barrel to thaw ice. The injection of live steam into the hydrant barrel through a hose is relatively quick, inexpensive, and effective. Jacobson[1] reports the thawing of hydrants in an emergency, in the average time of 3 min., by pouring concentrated sulphuric acid into the hydrant. The heat generated melts the ice before the acid reaches the valve. The acid is expelled from the hydrant when the valve is opened, and the acid is so diluted as to be harmless if properly handled. The first flush of water from the hydrant may be carried through a hose to the fire pumper where it is adequately diluted to prevent injury to personnel and equipment.

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[1] JACOBSON, E. E., "Rapid Emergency Method of Thawing Hydrants," *Water & Sewage Works*, April, 1946, p R-42.

CHAPTER XXIV

QUALITY OF WATER SUPPLIES

420. Considerations of Quality. Water to be used for a public water supply must be potable, *i.e.*, drinkable. Potable water should not be chemically pure, because water devoid of dissolved and suspended matter is both unpalatable and unhygienic. It is generally believed that the presence of certain minerals in water is essential to health[1] and, for this reason, certain waters are considered to be more healthful than others. Water that is ideal as a beverage may not be suitable for all other purposes; *e.g.*, it must be soft for bathing and washing, it must be free from dissolved gases if used in the manufacture of ice, it must be free from color if used in the manufacture of certain kinds of paper. Some desirable and some undesirable constituents of water for various purposes are listed in Table 74.

TABLE 74.—DESIRABLE AND UNDESIRABLE MINERAL MATTER IN WATER
(The amounts are stated in parts per million of the ion indicated. They are to be taken as suggestive only.)

Use	Desirable or permissible	Undesirable or objectionable
Domestic, drinking, and cooking	Ca Mg Li HCO_3 I 30 10 5 150 0.01	CO_3 SO_4 Cl Fe F 20 100 200 0.5 0.1
Washing ..	Na HCO_3 10 60	Ca Mg Cl SO_4 Fe 10 5 500 500 0.1
Laundry .	None	All
Irrigation	Ca Mg K 40 20	Na Cl SO_4 CO_3 B 10 100 200 10 0.2
Textile manufacturing ..	None or very little	Ca Mg SO_4 Cl HCO_3 Fe 10 5 100 100 200 0.1
Sugar manufacturing	None	Ca Mg SO_4 Cl HCO_3 Fe 20 10 20 20 100 0.1
Pottery manufacturing	Na HCO_3 50 200	SO_4 Cl 20 20
Steam boiler ...	None	Ca Mg SO_4 HCO_3 Cl 5 5 100 100 100

Chemically pure water is not found in nature because of the solvent power of water and the wide distribution of finely divided soluble matter in the atmosphere and in the ground. Substances commonly found in water in its natural state include oxygen, carbon dioxide, iron, salts of calcium and magnesium, and other constituents whose hygienic effects are not yet thoroughly understood. Some substances found in natural water supplies are listed in Table 75.

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[1] See also PETERS, J. B., "Body Water," C. C. Thomas, Springfield, Illinois, 1935.

TABLE 75.—SUBSTANCES COMMONLY FOUND IN NATURAL WATER SUPPLIES[1]

Meteorological Waters

Suspended: amorphous matter.
Dissolved: sulphites, oxygen, nitrogen, carbon dioxide, chlorides.
Colloidal or pseudosolution: none.

Surface Waters

Suspended: clay, mineral matter, algae, bacteria, organic matter, protozoa.
Dissolved: oxygen, nitrogen, carbon dioxide, coloring matter, organic acids, organic matter, ammonia, chlorides, nitrates, nitrites, methane, hydrogen sulphide, hydrogen.
Colloidal or pseudosolution: coloring matter, silica, organic acids, organic matter.

Ground Water

Suspended: clay, fungi.
Dissolved: bicarbonates, carbonates, sulphates, chlorides, and nitrates of calcium, magnesium, sodium, and potassium. Bicarbonates, sulphates, and hydroxides of iron. Sodium chloride, compounds of manganese, carbon dioxide, oxygen, nitrogen, hydrogen sulphide, and hydrogen.
Colloidal or pseudosolution: silica, alumina, iron oxide.

[1] This table does not present all the substances found in natural waters, and the division of matter between the various sources is not sharply fixed.

Many natural waters, besides containing certain undesirable quantities of dissolved and suspended matters, are polluted by drainage from populated regions. Nevertheless it is possible, through purification processes, to change the quality of most natural waters to render them safe and satisfactory for use by the public. The limiting concentration of undesirable substances permissible in a water being considered as a source for a public supply is determined more by hygienic, esthetic, and economic considerations than by the difficulties of purification. There are, however, substances such as oil and sodium chloride that cannot be satisfactorily removed to permit the use of the water as a public water supply.

The quality of a proposed source for a public water supply should be judged on the basis of a sanitary survey of the source, a complete sanitary analysis made of samples taken periodically over a long period of time, and a consideration of the economic and esthetic factors involved in the use of the water.

421. Quality of Natural Waters. Water in nature may be classified as meteorological water, surface water, or ground water according to the nature of its natural environment. The nature of the environment affects the quality of the water, as indicated in Table 75.

Meteorological waters are precipitated from the atmosphere as rain, snow, hail, etc. They are seldom used as a source of public water supply because of the difficulty of collection. Where these waters are to be used, the precipitation is caught on prepared catchment areas and is stored in reservoirs. If not properly cared for by the removal of debris from the catchment area and the prevention of growths in the reservoir, an otherwise satisfactory water may be ruined as a source of supply. Generally speaking, the sanitary quality of meteorological water is satisfactory for a public water supply without purification.

Surface waters are natural waters found in lakes and streams. The quality of a surface water is determined mainly by the character of the watershed over which it has run. If the watershed is inhabited, the water is more likely to be polluted than in the case of either meteorological or ground waters. A surface water that is satisfactory as a source of a public water supply should be saturated with dissolved oxygen and should have a chloride content no higher than the normal for the region. Surface waters running over boggy ground are likely to be highly colored. Clay soils will cause turbidity. Water that has run off cultivated land is likely to be high in bacteria. During the summer the temperature of surface water will probably be high, unless taken from a deep lake. Surface water may contain dissolved minerals characteristic of the region. Iron may be present in organic or colloidal form. The total solids of surface waters may be high, but the fixed solids will be relatively low because of the amount of dissolved organic matter present. The growth of vegetation may become so dense in surface waters as to increase the organic content and to affect the odor, taste, and color of the water. Generally speaking, the sanitary quality of surface water is unsafe for use as a public water supply without some form of treatment.

Ground waters are natural waters that are found below the surface of the ground. Water that has percolated through the ground may be returned to the surface through springs, thus imparting the characteristics of ground water to surface water. Water that has taken on the characteristics of surface water may percolate, unchanged, through underground strata, thus imparting the characteristics of surface water to ground water. The line between the characteristics of surface waters and of ground waters cannot be sharply drawn.

Typical ground waters have distinguishing characteristics. Groundwater temperatures are usually constant throughout the year and are 2 to 3°F. higher than the average yearly air temperature of the region, increasing in temperature about 1°F. for each additional 64 ft. of depth, with variations between 50 and 100 ft. In percolating through the soil, ground waters absorb carbonic acid and organic acids from decaying organic matter, and they usually contain carbon dioxide and nitrogen. The acidity and the carbon dioxide increase the solubility of other matters, particularly mineral matter, with the result that the amount of dissolved matters in ground water is high. The oxygen dissolved in the water before it percolates through the ground is usually exhausted in the ground by the oxidation of the dissolved organic matter, with the result that ground waters are usually devoid of, or deficient in, dissloved oxygen. Thus, certain substances that are insoluble in the presence of oxygen are held in solution in ground waters.

Water in percolating through underground strata dissolves from the rocks the alkalies and alkaline earths in the form of bicarbonates, and silica may be held in colloidal suspension. Such substances as carbonates of calcium and magnesium cause hardness when dissolved in water. Bicarbonates of these elements are known as alkalinity when present in solution. The colloidal suspension of silica, alumina, and iron oxide is enhanced by the presence of alkalinity.

Relatively high alkalinity and low hardness, together with the presence of silica, alumina, and possibly iron and manganese, are characteristic of waters that have been in contact with granites, basalts, and gneisses, *i.e.*, the primary rocks. Water in contact with dolomites, limestones, sandstones, and shales, *i.e.*, the so-called secondary rocks, become impregnated with bicarbonates of calcium and magnesium and with chlorides.

Generally speaking, the sanitary quality of ground waters is satisfactory for use as a public water supply without purification, but the amount of dissolved minerals and other characteristics of the water may require its treatment for domestic and industrial purposes.

422. Calcium, Magnesium, and Sodium Salts. Hardness, alkalinity, and salinity in water are caused by the presence of dissolved mineral matter, primarily by the compounds of calcium, magnesium, and sodium. Although the presence of these substances in most natural waters has no sanitary significance, they may have so detrimental an effect on the use of the water for domestic and industrial purposes as to render it unfit as a public water supply.

Compounds of calcium, which are the most commonly found dissolved mineral substances in natural waters, are supplied from three principal sources: limestone, gypsum, and calcium chloride. Limestone, $CaCo_3$, is soluble in water to the extent of about 13 p.p.m. If carbon dioxide is present in solution, the solubility of the calcium carbonate may be increased to 1,000 p.p.m. Gypsum, $CaSO_4$, is soluble to the extent of

2,000 p.p.m.; and calcium chloride, $CaCl_2$, is so highly soluble as to render water unfit for drinking purposes long before saturation is approached. Magnesium compounds are found in natural waters in concentrations from 5 to 20 per cent of the amount of calcium. Sodium is found most commonly in deep ground waters, or at higher levels near the sea coast, and is usually in the form of sodium chloride.

423. Hardness. Hardness is caused by the solution in the water of carbonates and sulphates of calcium and magnesium. The chlorides and nitrates of these two elements and sometimes of iron and aluminum are effective to a lesser degree in causing hardness. Temporary hardness is that portion of the hardness which can be removed by boiling. It constitutes, in the main, all the carbonate hardness, because the boiling drives off the dissolved carbon dioxide, thus rendering the carbonates insoluble. Permanent hardness is that portion of the hardness which remains after the water has been boiled. It is caused primarily by the sulphates of calcium and magnesium. It can be removed only by chemical means. Temporary hardness is not, however, equivalent to carbonate hardness, nor is permanent hardness equivalent to sulphate hardness. Total hardness is expressed in various ways, the standard American waterworks practice being in parts per million by weight, in terms of calcium carbonate, $CaCO_3$.

The U.S. Geological Survey has classified waters of various degrees of hardness, as shown in Table 76. The total hardness of natural water supplies used as sources of drinking water, either with or without softening, range from slightly below 10 p.p.m. to about 1,800 p.p.m.

TABLE 76. HARDNESS OF WATER

Class	1	2	3	4
Hardness, p.p.m.	0–55	56–100	101–200	201–500
Degree of hardness	Soft	Slightly hard	Moderately hard	Very hard

The effects of hardness in water include the inhibition of lathering by soap, the formation of an insoluble precipitate or curd with soap, the formation of "soft scale" in boilers due to the presence of calcium carbonate, the formation of "hard scale" due to the presence of sulphate hardness, and other difficulties. Soft scale is readily soluble in acid. Hard scale may be difficult to remove.

424. Alkalinity and Salinity. Alkalinity of natural water represents its content of carbonates, bicarbonates, hydroxides, and, occasionally, silicates and phosphates. Alkalinity is most commonly found in natural waters in the form of carbonate of soda and as bicarbonate of calcium and bicarbonate of magnesium. Caustic alkalinity, caused by hydroxides, is an undesirable characteristic seldom found in natural waters.

Natural waters are normally alkaline, acidity being an undesirable characteristic mainly due to the corrosive activity of acid waters. Neither acidity nor alkalinity has sanitary significance in natural water.

Salinity in water is caused by the presence of carbonates, sulphates, nitrates, and chlorides of calcium and magnesium and by the sulphates and chlorides of sodium.

425. Chloride. Chlorides are present in natural waters mainly as salts of sodium and may indicate either the degree of brackishness or, if higher than normal for the natural waters of the region, pollution by sewage or by saline industrial wastes. Chlorides must not be confused with free chlorine, a gas used for disinfection and rarely found in natural untreated water.

To avoid unpalatability the chloride content of drinking water should not exceed 250 p.p.m.

426. Iron and Manganese. Iron and manganese are objectionable in a public water supply because they cause stains on plumbing fixtures and on clothing in the laundry, they may cause tastes and odors, and they offer difficulties to some manufacturing processes. The presence of organisms whose life processes depend on these elements or their compounds causes tastes and odors in the water and may result in what is known as "red water." Those organisms depending on the presence of iron are sometimes known as "iron bacteria." Compounds of iron, as found in surface waters, are usually in the form of ferrous bicarbonate or ferrous or ferric sulphate. Ferrous bicarbonate is formed by the reduction of ferric oxide, the mineral ore, to ferrous oxide and the combination of the latter with carbon dioxide present in the water. Ferrous bicarbonate, $Fe(HCO_3)_2$, is soluble only in the absence of oxygen. When oxygen is available, the ferrous bicarbonate is quickly oxidized to ferric hydroxide, $Fe(OH)_3$, which is highly insoluble and quickly precipitates. The presence of iron is common in ground water because of the wide distribution of iron as hematite in nature and its solution in water containing carbonic acid. The sulphates of iron frequently result from contact of the water with coal and iron ores. The quantity of iron in ground water may increase after the development of a well owing to the accumulation of organic matter which, on decomposition, will produce acid that dissolves an increasing amount of iron from the soil.

Most ground waters, even in iron-bearing regions, contain less than 5 p.p.m. of iron, some falling into the range between 5 and 15 p.p.m., with a maximum of 40 to 50 p.p.m. for acid mine waters.

Manganese behaves similarly to iron in its reactions in natural waters.

As it is not so widely distributed as iron in nature, its presence in water may not be recognized through its confusion with iron. In general, the maximum concentration of iron or of manganese, or the combination of the two, which is permissible in a water supply is about 0.3 to 0.5 p.p.m.

427. Lead, Copper, Zinc, and Tin. Lead is generally considered undesirable in water because of its tendency to accumulate in the human body, resulting in lead poisoning, although in some proposed standards of quality of water a maximum concentration of lead of 0.1 p.p.m. is permitted. Extremely small quantities of copper and of zinc are sometimes considered to be hygienically desirable, but concentrations greater than 0.3 p.p.m. of copper and 5.0 p.p.m. of zinc are undesirable.[1]

The amount of copper, zinc, lead, and tin normally found in natural water supplies is practically insignificant. However, water flowing through or standing in conduits or reservoirs containing these metals or coated with such metallic paints may absorb traces of the metals. In general, it is considered that dangers from this source are negligible, except possibly in the case of lead. Copper in water sometimes results from the treatment of a reservoir with copper sulphate. It is the general consensus of opinion that the quantities thus used have no detrimental effect on the human organism.[2]

428. Other Minerals. Sulphur may exist in water as sulphates or sulphides, originating from contact with iron pyrites or other ores. It may make its presence manifest by a decided odor of hydrogen sulphide, rendering the water esthetically unfit as a public water supply. Hydrogen sulphide is objectionable also because of its corrosiveness in water.

Aluminum and ammonium do not ordinarily occur in natural waters in sufficient quantities to impose a problem for public water supplies.

Iodides, bromides, fluorides, phosphates, etc., are rarely found in sufficient quantity to render natural waters unfit for public use.[3] The presence of iodides in water, in a concentration up to 1 p.p.m., is generally considered to be desirable to prevent goiter. Fluorides have been found in the natural waters of the Midwestern and Southwestern states, generally in relatively low concentrations but in some cases as high as 4.0 p.p.m.[4] Concentrations higher than 1.0 p.p.m. may be expected to cause mottling of the enamel of teeth in infancy. On the other hand, attention has been called to the need for a minimum fluoride content in drinking water to inhibit dental caries.[1]

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[1] See *Jour. A.W.W.A.*, January, 1943, p. 98; *ibid.*, February, 1943, p. 135; and *ibid.*, March, 1946, p. 362. See also SCHNEIDER, W. G., "Copper and Health," *Jour. New Engl. Water Works Assoc.*, Vol. 44, p. 435, 1930; and PALMER, R. M., "The Presence of Zinc in Water," *Proc. Am. Soc. Sanitary Eng.*, 1930–1931, p. 364.

[2] See also MALLORY, F. B., "The Poisonous Effects of Copper," *Jour. New Engl. Water Works Assoc.*, Vol. 41, p. 27, 1927.

[3] See also LOEB, L. B., "Radium Poisoning in Water," *Jour. A.W.W.A.*, April, 1934, p. 461.

[4] See also DE WITT, D. J., and M. S. NICHOLS, "Fluoride Content of Some Wisconsin Waters," *Jour. A.W.W.A.*, July, 1937, pp. 980–997.

Silica is present in many natural waters in concentrations between 2 and 60 p.p.m. and to a maximum of over 110 p.p.m. It is objectionable because of the scale it forms in steam boilers.

Arsenic has been found in natural waters in one recorded instance[2] in sufficient concentration to cause fatality.

429. Methane and Carbon Dioxide. Methane has been found in ground waters or has been released from wells in sufficient quantities to cause violent explosions.[3] The gas is colorless, odorless, and has a specific gravity of 0.554 that of air at 0°C. In a mixture with air at a concentration between 5 and 15 per cent it is explosive. The methane may rise through the well between the casing and the drop pipe, or it may escape from solution in the water as it is pumped from the well to mix with the air of the pumping station or other building in which the well is located. Where the presence of methane is below a concentration of 1.4 p.p.m., or 0.28 cu. ft. per thousand gallons at a temperature of 57°F., sufficient methane cannot escape from the water to form a 5 per cent mixture with the surrounding air. It can be excluded from the building by closing the top of the well casing and leading a vent pipe from inside the casing to the outer air. The vent pipe should be at least 2 in. in diameter and should be protected against clogging by frost or other causes. Precautions must be taken against the presence of methane in the air of a building in concentrations between 5 and 15 per cent in order to avoid the presence of an explosive mixture.

The concentration of carbon dioxide is of importance in a water supply because of (1) the desirable taste that it imparts, (2) its effect in increasing the solubility of many minerals in water, and (3) the corrosiveness resulting from its presence. A desirable balance between the concentration of carbon dioxide and calcium carbonate, spoken of as the carbonate balance (see page 558), should be maintained in a water supply.

430. Ionic Concentration. The hydrogen-ion concentration in water in connection with its mineral content may be of physiological importance because the metabolism of an individual accustomed to a water of a particular pH and mineral content may be disturbed, with consequent digestive disorders, when changing to a water with a different combination of pH and mineral content, despite the fact that both waters are potable and salubrious to those habitually consuming them. This fact serves to explain, in part, digestive disorders experienced by travelers sometimes attributed to a change of drinking water.

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[1] See also Cox, J. G., "New Knowledge of Fluorine in Relation to Dental Caries," *Jour. A.W.W.A.*, November, 1939, p 1926.

[2] See also Wyllie, J., "An Investigation of the Source of Arsenic in Well Water," *Can. Pub. Health Jour.*, Vol. 28, p. 128, 1937.

[3] See also Buswell, A. M., and C. C. Larson, "Methane in Ground Waters," *Jour. A.W W.A.*, December, 1937, p. 1978.

431. Organic Life in Water. Living organisms can be divided into three classes according to their size, as macroscopic, microscopic, and bacteria. Macroscopic organisms are so large that the species can be distinguished by the unaided eye; a fish or a snail would be an example. Microscopic organisms are so small that the aid of a microscope is necessary to distinguish the species, *e.g.*, Daphnia or Asterionella, as shown in Fig. 220. Bacteria are so small that even the aid of the microscope is insufficient to permit the complete identification of species. This must be done with the aid of stains, reaction to certain nutrient media or other environmental conditions. Natural waters, particularly surface waters, may abound in various kinds of organic life, some of which are detrimental to the use of the water for a public supply.

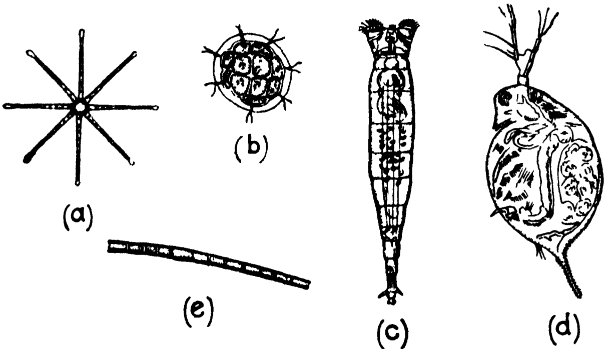

FIG. 220 Microscopic organisms (*a*), Astgrionella gracillima (colorless) × 188; (*b*), Pandorina (color, green) × 385, (*c*), Rotifer citrinus × 170; (*d*), Daphnia × 35; (*e*), Crenothrix × 500

432. Bacteria in Water. Bacteria vary greatly in size. The largest known bacterium is *B. butschlii*, which is about 60μ in length and 5μ in diameter. One μ, called a micron, is approximately 1/25,000 in. *Dialister pneumosintes* is one of the smallest organisms that has been described. It is 0.15μ in diameter and 0.3μ in length. The various kinds of bacteria are almost numberless, the vast majority being either beneficial or harmless to man. Those bacteria which cause disease in man are known as pathogenic. There are relatively few pathogenic bacteria, and but few of these can be carried by water.

Practically all pathogenic bacteria that can be borne by water are indigenous to the intestinal tract of warm-blooded animals, particularly man; unfortunately their isolation from natural water is difficult by laboratory means. Normally, water-borne pathogenic bacteria will not multiply in the unfavorable environment of natural water or laboratory culture media. There is, however, a harmless bacterium, known as *Escherichia coli,* normally present in the intestinal tract of warm-blooded animals, that is relatively easily isolated and identified. The presence of this organism in a sample of water is taken to indicate the presence of fecal matter in the water and is a warning that pathogenic bacteria indigenous to the intestinal environment may also be present. Hence, the importance of the *E. coli* test. It is the only specific form of bacterium for which a search is made in studying the sanitary quality of a water supply, and upon its presence or absence alone is often based the opinion of the safety of the water for drinking purposes.

433. Water-borne Organisms Causing Disease. Practically the only pathogenic bacteria that can survive in water, without protection in particulate matter or other physical substance, long enough to transmit disease to man are *Eberthella typhi,* causing typhoid fever; *Vibrio comma,* causing Asiatic cholera; and various coliform organisms causing bacterial dysentery. *Endamoeba histolytica* is a microscopic organism, classed as protozoa, which may be water-borne and which causes amoebic dysentery, or amebiasis, in man.

There is some evidence that the virus of poliomyelitis may be water-borne. Epidemics of infectious hepatitis, an infection of the liver sometimes called infectious jaundice from its characteristic symptom, have been traced to infected water. It is known that "swimmer's itch" and other skin infections designated as Schistosome dermatitis affect bathers. The causative agent is said to be the cercaria, the free-swimming larval stage of certain parasitic worms of the family Schistosomatidae.

434. Iron and Sulphur Bacteria. Many different microorganisms grow in water and add to the difficulties of its purification. Among these undesirable organisms may be included[1] (1) iron bacteria, (2) sulphur bacteria, (3) sulphate-reducing bacteria, and (4) nonspecific bacteria causing iron transformations. Difficulties caused by these organisms include fouling of the water supply and corrosion or clogging of the distributing pipes.

Iron bacteria include *Crenothrix, Leptothrix, Spirophyllum, Gallionella,* and others. Concentrations of iron as low as 0.1 p.p.m. will support the life of such organisms in water. The organisms secrete iron and deposit ferric hydroxide. Iron bacteria oxidize ferrous iron to ferric iron and precipitate ferric hydrate. Some sulphur bacteria increase acidity, and this tends to dissolve iron; under specific conditions insoluble iron sulphate may be produced. Sulphate-reducing bacteria may cause the precipitation of iron by the production of sulphide from the reduction of sulphate. These bacteria also effect the aerobic corrosion of iron. Other bacteria cause changes in the iron content of water by altering the environmental conditions, such as (1) changes in reaction, (2) changes in degree of oxidation or reduction, (3) production and decomposition of compounds of iron, and (4) changes in the carbon dioxide content of the water. No single set of methods can be used to determine the nature of these organisms or to identify them in the laboratory. Their control is sometimes effected by the use of chlorine, copper sulphate, and the removal of their food supply, such as dissolved iron, nitrogen, or sulphur.

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[1] See also STARKEY, R. S., "Transformations of Iron by Bacteria in Water," *Jour. A.W.W.A.*, October, 1945, p. 963.

435. Microscopic Organisms. Microscopic organisms are found principally in surface waters; although a few fungi and other organisms are occasionally found in ground waters or in waters stored in the absence of light. The principal objection to the presence of microscopic organisms in a public water supply are the tastes and odors they produce. Some microscopic plants and animals found in natural waters, in purification plants, and in distribution systems are listed in Table 77.

Microscopic organisms vary in size so that counting them is insufficient to give a correct conception of the concentration of organic life in the water. The crowding of animals in a cage would be much greater if the cage contained 100 elephants than if it contained 100 field mice. To overcome this difficulty, microscopical analyses are reported both in terms of the number of organisms present per cubic centimeter and in standard units per cubic centimeter. The number per cubic centimeter is determined by counting the organisms found in a small volume of water into which the organisms from a larger known volume of an original sample of the water may have been concentrated. A standard unit is represented by a square 20μ on a side. Since many organisms are approximately the same size, the number of these organisms per unit volume can be approximated by counting those in a counting cell and multiplying by a factor dependent on the size of the organism counted.

It has been found[1] that when cysts of *E. histolytica* are exposed to solutions of chlorine or certain chloramines (see page 524) the cysts are penetrated and killed.

436. Macroscopic Organisms. The macroscopic organisms most commonly causing difficulties in natural waters include cattails, hyacinth, eel grass, and sea weeds. None of these organisms is desirable in water-supply sources, but their presence is sometimes tolerated because of the

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[1] See CHANG, SHIH LU, "Destruction of Micro-organisms," *Jour. A.W.W.A.*, November, 1944, p. 1192.

TABLE 77.—PARTIAL LIST OF MICROORGANISMS THAT ARE TROUBLESOME IN WATER SUPPLIES, TOGETHER WITH A STATEMENT OF THE AMOUNT OF CHEMICALS REQUIRED FOR THEIR EXTERMINATION[1]

Organism	Odor	$CuSO_4$, p.p.m.	Cl_2, p.p.m.	Organism	Odor	$CuSO_4$, p.p.m.	Cl_2, p.p.m.
Diatoms:				Cyanophyceae:			
Asterionella[2]	AGF	0.12 to 0.20	0.5 to 1.0	Anabaena[2]	MGV	0.12	0.5 to 1
Cyclotella[2,3]	fA		1	Aphanizomenon[2]	MGV	0.12 to 0.5	0.5 to 1
Diatoma[3]	fA			Clathrocystis	SGV	0.12 to 0.25	0.5 to 1
Fragalaria[3]		0.25		Coleosphaerium[2]	SG	0.2 to 0.33	0.5 to 1
Melosira[2,3,4]				Cylindospermum	G	0.12	
Meridion	A			Microcystis		0.2	
Navicula[3]		0.07		Oscillaria[3]		0.2 to 0.5	
Nitzchia[4]		0.5		Rivularia	MG		
Synedra[2,3]	fA	0.36 to 0.5	1	Schizomycetes:			
Stephanodiscus		0.33		Beggiatoa[3]	vO, D	5	
Tabellaria	AGF	0.12 to 0.5	0.5 to 1	Cladothrix		0.2	
Chlorophyceae:				Crenothrix[3]	vO, D	0.33 to 0.5	0.5
Cladophora		0.5		Leptothrix			
Closterium		0.17		Sphaerotilus natans	vO, D	0.4	
Coealstrum[3]		0.05 to 0.33		Thiothrix[2,3]	vO, D		0.5 to 1
Conferva[3]		0.25		Fungus:			
Desmidium		2.0		Leptomitus		0.4	
Dictyosphaerium	GNF		0.5 to 1	Saprolegnia		0.18	
Draparnaldia		0.33		Miscellaneous plants			
Eudorina	fF	10		Chara[3]		0.1 to 0.5	
Entomophora		0.5		Nitella flexilis[6]			
Gloeocystis	O			Patomogeton[3]		0.3 to 0.8	
Hydrodictyon	vO	0.1		Protozoa.			
Microspora		0.4		Busaria	im, sm, F	0.33	0.3 to 1
Palmella		2.0		Chlamydomonas		0.5	
Pandorina	fF	10		Cryptomonas	cv	0.5	
Raphidium		1.0		Dinobryon[2]	AV'F	0.18	
Scenedesmus		1.0		Euglena		0.5	
Spirogyra[3]		0.12		Glenodinium	F	0.5	
Staurastrium		1.5		Mallomonas	AV'F	0.5	
Tetrastrum			1	Peridinium	F'	0.5 to 2	
Ulothrix		0.2		Synura[2]	CM'FB	0.12 to 0.25	0.3 to 1
Volvox[2]		0.25	0.3 to 1	Uroglena[2]	FO'C'	0.05 to 0.2	0.3 to 1
Zygnema		0.5		Crustaceae:			
				Cyclops[2,3,4]			1 to 3
				Daphnia			1 to 3

A = aromatic. B = bitter. C = cucumber C' = cod-liver oil cv = candied violets. D = decayed. E = earthy. F = fishy. F' = fishy, like clamshells G = grassy im = Irish moss. M = moldy M' = muskmelon N = nasturtium. O = offensive O' = oily S = sweet sm = salt marsh V = vile f = faint v = very.

[1] PURDY, W. C., "Methods for the Biological Examination of Water," *Proc. 9th Ann. Water Works School, Univ. Kan.* 1931

[2] These organisms have been affected by chlorine, producing a characteristic odor. In many cases they have been controlled by doses of chlorine ranging from 0.3 to 3.0 p.p.m., depending largely on the amount of organisms present.

[3] These organisms have caused troubles other than odor.

[4] These may be controlled by excess lime. 5 p.p.m. in the case of diatoms.

[5] Medicinal with chlorine.

[6] Objectionable.

difficulty of removing them. Tastes, odors, and color caused by them may be removed in the process of purification. They cause no specific disease in man.

Macroscopic animals include fish, worms, crustacea, and insects. Few of these cause anything but minor troubles in water-supply sources, except that they must be prevented from entering the distribution system by the installation of screens or the use of sand filters.

437. Limnology. Limnology is the science that deals with fresh waters other than running streams. The quality of water from such sources is affected by physical features, aquatic life, climate, location, and other factors. The physical features include the depth of the water, the shape of the bottom, and the character of the banks and the shore line. The aquatic life includes microorganisms and, to a slight degree, bacteria. Climate is an important factor affecting the quality of the water supply through temperature, wind, precipitation, evaporation, and light. The location of the body of water affects its quality through the proximity of habitations and the use of the water for recreational and other purposes.

Natural bodies of fresh water, other than flowing water, can be divided into three groups depending for their differentiation on their depth and the biologic life in them. A lake may be considered as a body of water so large and so deep as to be subject to what is known as the spring and fall turnovers. A pond is a body of water that may contain a permanent biologic life but is so shallow that its depth is not a factor in variations in the character or concentration of aquatic life in it. A puddle is a temporary body of water that contains no permanent form of aquatic life. A reservoir may possess any one of these characteristics, dependent on its size and environment.

438. Rheology. Rheology is the science that deals with the properties and characteristics of streams. Physical, chemical, and biological factors affect the quality of the water in natural streams somewhat as they do in relatively quiescent bodies of water. In a flowing stream, however, the changes in environmental conditions and in the quality of water are so sudden as materially to affect the nature of the plankton. Such changes, resulting in the self-purification of a polluted stream, are effected principally by sedimentation, dilution, aeration, light, and biological activities. The degree of self-purification of a polluted running stream before it reaches the waterworks intake should be known to the engineer before recommending the stream as a source of water supply. The degree of self-purification of a running stream may be studied by plotting graphically the concentration of various significant constituents of the stream, particularly dissolved oxygen and bacteria, against the distance along the stream away from a fixed point. The increase in the percentage of saturation with dissolved oxygen and the diminution of the bacterial count are valuable measures of the improving sanitary quality of the water.

439. Factors Affecting Organic Life in Water. Aquatic life is affected by physical, chemical, and biological factors. The physical factors include temperature, currents, density, and intensity and penetration of light. The chemical factors include alkalinity, dissolved oxygen and other gases, and the food supply. The biological conditions include the presence and numbers of organisms and their distribution.

440. Temperature. Temperature is probably one of the most important of the physical factors affecting aquatic life, because the life of all organsims is limited by the range of temperature of the water. The growth of microorganisms that commonly affect the quality of water supplies practically ceases at temperatures below 35°F. and above 90°F. The optimum temperature for growth ranges between 60 and 80°, depending on the organism affected and other conditions. Temperature is also an important factor in creating currents and in affecting the density and viscosity of the water.[1]

441. Stratification and Circulation Due to Temperature. During certain seasons of the year the waters of a lake are stratified in accordance with temperature values, the difference of temperature between strata being marked. If, under conditions of stratification, the temperature at various depths in a lake is plotted graphically against depth, a depth will be found at which there is a marked break in the slope of the plotted line. The zone of depth where this break occurs is known as the *thermocline.* In general, the zone above the thermocline is a zone of circulation due to convection currents, and the zone below it is a zone of stagnation with diminished biological activity. The thermocline rises in winter and descends in summer.

In the spring and in the fall the waters of a lake above the thermocline turn over as a result of changing density in the water at the surface. The effect of these turnovers is important on the distribution of food and of biologic life. During the winter the surface layer of water is cold or is turned to ice with its temperature below 32°F. As the surface water becomes warmer in the spring, its density increases as it approaches 39.2°F. This heavier surface water sinks to the thermocline, causing the spring turnover. During the summer the surface layer of water is warm. As the surface water is cooled in the fall, its density increases as it approaches 39.2°F. This heavier surface water sinks to the thermocline, causing the fall turnover.

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[1] The temperatures of underground and surface waters in the United States are discussed in *U.S. Geol. Survey, Water Supply Paper* 520-F, 1925.

442. Currents. Currents are caused by changes in temperature, wind, incoming water, movement of boats, etc. The blowing of the wind creates waves which may result in stirring to depths as great as 40 ft., as experienced in the Great Lakes. A long-continued blow from one direction may result in the complete turnover of the lake, above the thermocline, greatly affecting turbidity and the distribution of microorganisms and their food supply. Since most microorganisms depend on currents for their own distribution and for the distribution of their food supply, the existence of gentle currents is an important factor affecting the growth and concentration of microorganisms.

443. Density and Viscosity. Density and viscosity affect the ability of certain microorganisms to float or to remain suspended. Some organisms, including most diatoms, are heavier than water and would sink were it not for upward currents and other conditions. Some organisms prevent themselves from sinking by the development of oil sacks, by entraining gases, or by increasing their surface area, through change of shape or size, to compensate for changes in density of the water or of themselves. Temperature is practically the only factor that materially affects the density and viscosity of water. Dissolved solids are negligible in their effect on the density and viscosity of water available for a public water supply.

444. Light. Light is necessary for the metabolism of all algae and most protozoa. It is detrimental to the growth of fungi. The growth of certain organisms that require light ,may be inhibited or destroyed by too great an intensity of light. Hence, the penetration of water by light is important in affecting the type of organism to be found at various depths. The penetration of light into water is dependent on the position of the sun and on the turbidity of the water and is limited by the absorption of the light by the water. The clearer the water, the greater the penetration by light and the greater the depth at which organisms depending upon photosynthesis are to be found. The absorption of light by water is so great that in the clearest water only about 25 per cent of the light remains at a depth of 30 ft. More than half the sunlight is absorbed at the surface. Because of the need for light by most organisms the covering of a reservoir will diminish organic growth. Unfortunately the absence of light, together with an abundant food supply, may encourage the growth of fungi.

445. Food Supply. The food supply of microscopic organisms depends on dissolved oxygen and suspended organic and inorganic matter, bacteria, other microorganisms, and dissolved gases. Alkalinity, dissolved gases, and food supply are closely related factors affecting organic growth in water. The inorganic matter normally encountered in natural waters furnishes part of the food supply for organisms in the water. Dissolved gases such as oxygen, carbon dioxide, and nitrogen constitute a part of the food supply. Other nutritive gases that may be present include methane, carbon dioxide, and hydrogen sulphide.

446. Dissolved Gases. The relation between dissolved oxygen and carbon dioxide may indicate the condition of the quality of the water or of changes that are occurring in it. Dissolved oxygen in water is derived from the atmosphere, and pure natural surface waters are normally saturated with it.

Carbon dioxide in water may be a result of decomposition of organic matter or of the metabolism of some organisms. In this action dissolved oxygen is converted to carbon dioxide. Hence, in general, as carbon dioxide increases, dissolved oxygen decreases. A high carbon dioxide content may, therefore, indicate biological activities, and high dissolved oxygen may indicate a high degree of purity of the water supply. This is not always true, however, as a few organisms secrete oxygen which may result in a supersaturation of the water with dissolved oxygen; or ground water high in carbon dioxide may enter the lake from submerged springs, thus increasing the carbon dioxide content of the water without affecting its sanitary quality. The amount of dissolved gases present for 100 per cent saturation is dependent on the temperature and on the presence of the gas in the atmosphere. At 0°C. and a pressure of 760 mm. of mercury, water will dissolve, from the air, 14.66 p.p.m. of oxygen; at 30°C. it will dissolve 7.63 p.p.m. Surface waters, under normal conditions of the air, may be expected to dissolve no more than 1 to 2 p.p.m. of carbon dioxide. Much larger amounts, over 100 p.p.m., may be dissolved in ground water under favorable conditions.

447. Biologic Growth. The kinds and numbers of microorganisms growing in surface waters vary with the season, with the food supply, and with other factors. In the plant kingdom schizomycetes and other fungi show the greatest independence to seasonal changes, to temperature, and to depth of growth in water. They may be found at all seasons at various depths but, being sensitive to light, are more frequently found in covered reservoirs, pipes, and near the bottom of lakes and reservoirs.

Algae follow regular seasonal variations. The food supply of the diatom is presumed to consist of carbon, nitrates, ammonia, silica, and mineral matter. These constituents are found in the lower strata of lakes, under conditions unfavorable to the growth of diatoms. The growth of diatoms at the surface is increased following the spring and fall turnovers which bring food to the surface where oxygen and a moderate intensity of light are available. Diatoms grow most abundantly near the surface, although they will be found at all depths above the thermocline.

There are, therefore, two seasonal increases in the growth of diatomaceae, the spring and the fall. Diatoms are found at temperatures between 35 and 75°F., the optimum temperatures being between 40 and 60°. Chlorophyceae are most abundant in midsummer at 60 to 80°, and cyanophyceae in the early fall at 70 to 80°. Chlorophyceae and cyanophyceae are more buoyant than diatoms and may contain oil globules or gas bubbles to aid their buoyancy. They are, therefore, found more frequently near the surface and may even form a scum thereon. No fixed rule can be stated as to their general location, as variations in both seasonal and depth distribution are to be found.

Protozoa, as a group, do not show any marked seasonal maxima, because of the different characteristics of the organisms in the group. They are most commonly found near the surface; some form a scum; and other genera, avoiding bright sunlight, are most abundant at depths of 2 to 3 ft. Different genera are to be found at all depths. The growth of protozoa is dependent more on the food supply than on the season, whereas plant growths show more distinctive seasonal variations. Rotifera are found at all seasons but are most abundant in the summer. Crustacea show variations in different bodies of water but have no marked maxima.

448. Protection of Sources. Wells and other works for the collection of ground water and reservoirs, intakes, and similar structures for the collection of surface water require protection against contamination. Pollution may travel underground for great distances, particularly through underground watercourses such as exist in limestone regions. In the construction of shallow wells in rural districts it is generally considered satisfactory to locate the well more than 100 ft. from the nearest known source of pollution and to protect the top of the well, by means of a watertight curbing and cover, from contamination by surface drainage. In the construction of deep wells for municipal water supplies the log of the well should be kept to record the location of strata supplying undesirable water. This is later cased off before the completion of the well. Additional precautions must be taken to prevent the pollution of the well from surface contamination. This is done by waterproofing the top of the well so that water cannot enter it and by providing drainage channels away from the top to prevent its becoming submerged.

The protection of the quality of a surface water supply can be divided into two parts, that furnished by the policing and care of the watershed and that furnished by the treatment of the water. Full dependence for the protection of a water supply cannot be placed on laws and regulations. The protection furnished by the treatment of the water is by far the more effective and, in most instances, can be depended on. Watershed protection depends on the policies adopted by the responsible authorities. Some permit unrestricted use of reservoirs for recreational purposes,[1] limiting protection to the prevention of gross pollution by sewage or industrial wastes and the purification of the water before distribution. Others depend on isolation of the reservoir and watershed, allowing none but accredited employees of the waterworks authority to trespass on either. Watersheds are forested to discourage trespassing, as a protection against erosion, to prevent silting of reservoirs, and to provide revenue.[2] All grades of restriction of use are to be found between extremes, each giving apparent satisfaction[3] locally.

Where complete isolation of the watershed is impossible, the enforcement of reasonable regulations and the making of routine laboratory analyses of the water will furnish information of changes in quality and will aid in controlling the operation of the purification plant to prevent dangerous pollution of the public water supply. Extreme vigilance should be devoted to the detection of the outbreak of water-borne diseases on or in the neighborhood of the watershed, and steps should be taken immediately to isolate the sources of infection. Where the watershed is large or is entirely beyond the control of the waterworks authorities, reliance cannot be placed on the continuous purity of the supply. Only purification will protect.

449. Deterioration in Quality. After treatment the quality of water may change as the result of various causes. In a distribution system the quality of the water may be affected by (1) corrosion of the pipes; (2) flushing, which dislodges foreign matter; (3) ingress of dirt and living organisms through leaks; (4) cross connections with polluted water; (5) aftergrowths of organisms, including bacteria, algae, schizomycetes, worms, and insects; and (6) solution of metals with which the water comes in contact. The deterioration of quality after purification may be subtle, discouraging, and more difficult to correct than the undesirable characteristics of the original raw water.[4]

450. Effect of Storage. Storage of water may either benefit or injure its quality. It is usually beneficial, however, as settleable matter will be removed by sedimentation, pathogenic bacteria and many others will die as a result of the unfavorable environment, decomposing organic matter will be oxidized, and color will be bleached out in open reservoirs. The storage of water may be detrimental to its quality because of the growth of microorganisms in it. This is particularly true in the storage of treated waters or in the storage of ground waters in open reservoirs, as a favorable environment with sufficient food is provided without the existence of predatory organisms to combat the increase of an undesirable species. The covering of the reservoir, used for the storage of treated water, will not assure the absence of difficulties, because the presence of desirable foods and the absence of light will encourage the growth of fungi, such as crenothrix, in the water. If a reservoir is to be covered, care should be taken to exclude insects, particularly the flying kind, which lay eggs in the water, resulting in the production of undesirable larvae therein.

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[1] See also DEVENDORF, EARL, "Recreational Use of Reservoirs," *Jour. A.W.W.A.*, April, 1939, p. 724.

[2] See also HEILMAN, J. M., "Forest Management for Newark," *Jour. A.W.W.A.*, January, 1947, p. 871.

[3] WESTON, A. D., "Sanitation of Watersheds," *Jour. New Engl. Water Works Assoc.*, 1935, p. 306.

[4] HOWARD, N. J., "Water Quality Deterioration in Distribution Systems," *Jour. A.W.W.A.*, 28, p. 1044, 1936.

The character of the catchment area materially affects the quality of impounded water because of the food supply and the type of organisms contributed to the reservoir. Swamps, ponds, puddles, or lakes may contribute sudden increases in organisms that are flushed into the reservoir by rains or freshets. Seasonal increases in organisms to which the environment or the swamp or higher body of water is favorable may result in the appearance of organisms in large numbers in the reservoir into which these higher bodies of water are drained.

The shape, depth, and area of a reservoir affect organic growths in stored water. A large surface area with a shallow depth is conducive to organic growth, since many troublesome organisms grow in the light near the surface. Both small and large reservoirs are susceptible to organic growth in the water. Irregularly shaped shore lines increase the surface area for equal capacity. They increase the growth of organisms along the shore, called littoral organisms; and they furnish coves, inlets, and protected harbors for growing organisms. A reservoir with a rough bottom is open to a similar objection in that the depressions furnish centers for organic growth.

The storage of water will sometimes affect its temperature undesirably, particularly that of ground water, which may be brought to the surface cool and palatable, only to be changed to a tepid, distasteful liquid after storage under a hot sun, in shallow reaches, over submerged vegetation.

451. Standards of Quality. The setting of a standard of quality to which all acceptable water supplies must conform has frequently been attempted and has, as frequently, been shown to be unsuitable for all conditions. It is impossible to set standards that will not exclude some potable waters or that may not include some unsatisfactory waters.

Nevertheless there has been a continued and insistent demand for the promulgation of acceptable standards of general application. The widespread voluntary adoption of the "Drinking Water Standards" adopted by the U.S. Public Health Service for the "Quality of Water on Interstate Carriers" is an indication of the general desire for some form of acceptable yardstick of water quality.

Standards of water quality can be divided into three types:[1] (1) for water of exceptionally great natural purity, (2) for pure waters from a restricted area, and (3) for limits of matters permitted in waters. An example of the first type is the "Standards of the Hygienic Laboratory of the University of Michigan."[2] An example of the second is "Haines Local Standards for Waters on the East Bank of the Delaware River near Philadelphia."[3] An example of the third type is the "Drinking Water Standards for Interstate Carriers" adopted by the U.S. Public Health Service,[4] as revised to Feb. 6, 1946. Extracts have been taken from these standards which are divided into four sections, as follows:

1. Definitions of Terms. A standard portion of water for bacteriological test is designated to be either 10 ml. or 100 ml. The standard samples in the bacteriological test may consist of five portions of either of the above sizes. If a sample collected for bacteriological examination is from a disinfected supply, the sample must be freed of any disinfecting agent within 20 min. from the time of collection.

2. As to Source and Protection. The water supply shall be (*a*) obtained from a source free from pollution, or (*b*) obtained from a source adequately purified by natural agencies, or (*c*) adequately protected by artificial treatment.

The water system in all its parts shall be free from all known sanitary defects, and health hazards shall be systematically removed at a rate satisfactory to the reporting agency and to the certifying authority.

3. As to Bacteriological Quality. The minimum number of samples to be collected from the distribution system and examined . . . is based upon the relationship of population served. Of all the standard 10-ml. portions examined per month not more than 10 per cent shall show the presence of organisms of the coliform group. Of all the 100-ml. portions examined per month not more than 60 per cent shall show the presence of organisms of the coliform group.

The procedure given, using a standard sample composed of five standard portions, provides for an estimation of the most probable number of coliform bacteria present in the sample as set forth in the following tabulation:

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[1] See HINMAN, J. J., "Standards of Quality of Water," *Jour. A.W.W.A.*, Vol. 7, p. 821, 1915.

[2] See *Public Health, Michigan*, Vol. 7, p. 9, January March, 1912.

[3] See HAINES, R., "Local Standards in Water Analysis," *Jour. Anal. and Applied Chem.*, Vol. 5, p. 289, May, 1891.

[4] See "Drinking Water Standards," *Jour. A.W.W.A.*, March, 1946, p. 361.

Number of portions		M.P.N.[1] of coliform bacteria per 100 ml.	
−	+	When five 10- ml. portions are examined	When five 100-ml. portions are examined
5	0	Less than 2.2	Less than 0.22
4	1	2.2	0.22
3	2	5.1	0.51
2	3	9.2	0.92
1	4	16.0	1.60
0	5	More than 16.0	More than 1.60

[1] M.P.N. means most probable number.

Physical characteristics. The turbidity shall not exceed 10 p.p.m. (silica scale), nor shall the color exceed 20 (cobalt scale). The water shall have no objectionable taste or odor.

Chemical Characteristics. The presence of lead in excess of 0.1 p.p.m., of fluoride in excess of 1.5 p.p.m., of arsenic in excess of 0.05 p.p.m., of selenium in excess of 0.05 p.p.m., of hexavalent chromium in excess of 0.05 p.p.m., shall constitute ground for rejection of the supply.

Salts of barium, hexavalent chromium, heavy metal glucosides, or other substances with deleterious physiological effects shall not be added to the system for water-treatment purposes.

The following chemical substances which may be present in natural or treated waters should preferably not occur in excess of the following concentrations when more suitable supplies are available in the judgment of the certifying authority: Cu should not exceed 3.0 p.p.m.; Fe and Mn, 0.3 p.p.m.; Mg, 125 p.p.m.; Zn, 15 p.p.m.; chloride 250 p.p.m.; sulphate, 250 p.p.m.

Phenolic compounds should not exceed 0.001 p.p.m. in terms of phenol.

Total solids should not exceed 500 p.p.m. for a water of good chemical quality. However, if such a water is not available a total solids content of 1,000 p.p.m. may be permitted.

452. Governmental Control over Water Quality. Many federal agencies are concerned with the quality of different types of public water supplies. These agencies, together with some information concerning their activities, are listed in Table 78.

The standards of quality for public water supplies served to the public by interstate carriers, adopted by the U.S. Public Health Service, which has jurisdiction over them, have had a highly beneficial effect on the quality of most municipal water supplies. Although the service has no legal jurisdiction over the quality of a local public water supply, the moral effect of the possible condemnation of the supply for use in interstate traffic is ordinarily sufficient to cause the waterworks authority to maintain a high standard of quality.

TABLE 78.—FEDERAL AGENCIES CONCERNED WITH URBAN AND RURAL WATER SUPPLIES

Agency	Urban	Rural	Agency	Urban	Rural
Bureau of Agricultural Engineering[1]	....	DS	International Joint Commission[3]	SR	
Forest Service[1]	....	DSC	Public Health Service[4]	SR	
Soil Conservation Service[1]	....	DSC	Corps of Engineers[5]	S	
Division of Grazing[2]	....	DSC	Emergency Conservation Work[6]	C	C
General Land Office[2]	DSR	DSR	National Resources Committee	S	S
National Park Service[2]	DSC	...	Panama Canal[6]	DSC	
Office of Indian Affairs	....	DSC	P.W.A.[6]	C	C
U.S. Geological Survey	DS	DS	R.F.C.[6]	C	
Boundary Commission, U.S., Mex.[3]	SCR	....	W.P.A.[6]	C	C

C means supervises construction.
D means collects data.
R means regulates nonfederal activity.
S means carries on studies and research.

[1] Department of Agriculture.
[2] Department of Interior.
[3] State Department.
[4] Federal Security Agency.
[5] U.S. Army.
[6] Independent agencies.

In most states and territories, control over the quality of the public water supplies of the region is vested by law in the state or territorial department of health. Under such laws the quality of the water, the method of operation of the waterworks, and projects for the alteration or extension of works providing water for public use are subject to review by the health authority. In most large cities, state supervision of the quality of the water supply is nominal, the responsibility being left in the hands of the local health authority or the waterworks department. Education and persuasion have been found more effective than threat and precept in the maintenance of satisfactory public water supplies.

453. Purposes and Methods of Water Purification. Public water supplies are purified primarily to protect the public health. They are purified also (1) for esthetic reasons, *e.g.*, in the removal of taste, odor, color, and turbidity; (2) for economic reasons, *e.g*, in softening to minimize laundry costs; (3) for industrial purposes, *e.g.*, in the preparation of water suitable for use in boilers; and (4) occasionally for other reasons such as for swimming pools, to reduce corrosiveness, or to combat goiter or fluorosis.

Water can be purified by screening, sedimentation, sedimentation with coagulation, filtration, and by other miscellaneous methods. It is to be noted that mechanical, chemical, and biological methods and equipment are involved in water purification and it is to be emphasized that the principal purpose of water purification is the protection of the public health through the prevention of water-borne diseases.

454. Water and the Public Health. Experience has shown that an improvement in the quality of the public water supply is followed by an improvement of the public health.[1]

Pollution of the public water supply and outbreaks of disease have been traced to various causes, among them being the contamination of surface watersheds by human activities, such as the construction or existence on them of camps, railroads, or highways; the leakage of polluted water into wells or other sources of underground water; the breakage of aqueducts or of submerged distribution pipes permitting the leakage into them of polluted water; the inundation of parts of a waterworks by floods of contaminated water; the relaxation of vigilance in the purification or sterilization of a public water supply; the dropping of carrion into reservoirs by birds; and the creation of breeding places for mosquitoes and other flying insects.

455. Diseases Borne by Water. Living matter that causes disease in man can be transported by various means, such as the hands; clothing; dust; sewage; refuse; food, either liquid or solid; and water. The variety of forms of living matter borne by water seems unlimited, but the varieties of pathogenic germs that can be carried by water are few. This is because of the unnatural environment offered to the pathogenic germs in natural waters. The natural environment of pathogenic germs is the human body and the bodies of other warm-blooded animals.

Among the best known specific diseases whose germs can be carried by water are typhoid fever, dysentery, and cholera. Typhoid fever is the most important of these, as measured by the number of deaths reported annually within the registration area in the United States.[2] Water has been suspected of carrying infectious hepatitis.[3] An epidemic of amebic dysentery that occurred in Chicago in 1933[4] attracted nationwide attention because of the spectacular nature of the outbreak and the wide distribution of the resulting cases over the country. The epidemic resulted from the pollution of ice water by sewage that escaped from a leaky and surcharged soil pipe passing over a water cooler.

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[1] See Sedgwick, W. T., and J. S. McNutt, *Jour. Infectious Diseases*, Vol 7, p. 489, 1910.

[2] See also Gorman, A E., and Abel Wolman, "Water-borne Outbreaks in the United States and Canada and Their Significance," with brief bibliography, *Jour. A.W.W.A.*, February, 1939, p. 225; and "Typhoid in the Large Cities in the United States in 1945," *Jour. Am. Med. Assoc.*, Vol. 131, p. 817, 1946.

[3] See also Neefe, J. R., and J. Stokes, Jr., *Jour. Am. Med. Assoc.*, Vol. 128, p. 1063, 1945.

[4] See "Epidemic Amebic Dysentery—The Chicago Outbreak of 1933," *Nat. Inst. Health Bull.* 166, 1936.

Epidemics of water-borne outbreaks of infectious hepatitis have been recorded,[1] and some evidence is available that poliomyelitis may be water-borne.[2]

The transportation by water of pathogenic bacteria causing other specific diseases will occur only under exceptional circumstances. Pathogenic bacteria, viruses, and other disease-causing organisms that can be water-borne are discussed on pages 392 and 393.

456. Other Diseases Due to Water. Diseases other than those of organic origin that may be related to the quality of the water supply include goiter, lead poisoning, other metallic poisonings, fluorosis or mottled enamel of the teeth, and nonspecific intestinal derangements.

The absence of iodine from water is assumed to be a contributing cause of goiter. "One of the most extraordinary facts of recent medicine and surgery is that goiter may be a factor in the occurrence of malignance or cancer."[3] The amount of iodine that may be required to maintain health is so small as to be undetectable in routine sanitary or mineral water analysis. It has been suggested that the addition of 0.1 lb. of sodium iodide per million gallons of water will prevent goiter, and this has been attempted in a few places, notably at Rochester, N.Y.[4]

Methemoglobinemia is a disease of infants, sometimes fatal, resulting from the ingestion of large amounts of nitrates found in some waters.[5] The disease results from feeding with milk that has been diluted with well water containing 180 to 619 p.p.m. of nitrates.

Lead is sufficiently soluble in water to be a menace to health. The dangers of lead poisoning are increased because of the tendency of the body to store rather than to eliminate lead. Small quantities of lead, if taken regularly, may be stored in the system until a sufficient quantity has been accumulated to produce the symptoms of lead poisoning. Evidence has been produced[6] to cast some doubt on the inability of the body to eliminate lead, but its subtle dangers are, nevertheless, quite real. Since 0.5 p.p.m. is considered dangerous to health, the use of lead in contact with potable water is considered, by some authorities, to be dangerous. Neither lead nor zinc paints should be used on surfaces in contact with potable water. Asphaltum, silicate, or enamel paints are preferable.[1]

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[1] NEEFE, JOHN R. and J. STOKES JR., "An Epidemic of Infectious Hepatitis Apparently Due to a Waterborne Agent," *Jour. Am. Med. Assoc.*, Vol. 128, p. 1063, 1945, and TRUSSELL, R. E., "Epidemiological Aspects of an Outbreak of Infectious Hepatitis," *Am. Jour. Hyg.*, Vol. 45, p 33, 1947.

[2] SCOBEY, RALPH R., "The Role of Water in the Etiology of Poliomyelitis," *Arch. Pediat.*, Vol. 63, p. 567, 1946.

[3] WESTON, W., "Iodine and Nutrition," *Am. Jour. Pub. Health*, Vol. 21, p. 715, 1931.

[4] See also "Water Works Practice," American Water Works Association, 1925, p. 275.

[5] FERRANT, MAURICE, "Methemoglobinemia," *Jour. Pediat.*, Vol. 29, p. 585, 1946.

[6] See also NEGUS, S. S., "Here's to Your Health," *Water Works Eng.*, Mar. 2, 1938, p. 266.

Other metals whose presence in too large a concentration may derange bodily metabolism sufficiently to cause disease include copper, zinc, and iron. In general, however, the physiological effect of the small quantities of these metals found in natural waters or dissolved from conduits or containers may be beneficial rather than injurious to health.

Fluorosis, a defect of the enamel of human teeth, is caused by the destructive action of fluorine in the water supply. Evidence points to the defect's being caused by the action of fluorine on the teeth during their eruption in childhood. Normally formed teeth have never been known to become mottled after eruption.[2] On the other hand, attention has been called to the need for a minimum fluoride content in drinking water to inhibit dental caries.[3] Some cities are adding fluorine to the public water supply for this purpose.[4]

Intestinal derangements may result from the presence in water of an excess of magnesium sulphate, of other alkaline substances, or of undesirable organic matter or even from the lack of certain substances to which the body has become habituated.[5] In recent years several outbreaks of gastroenteritis have occurred which, from certain aspects, appeared to be water-borne but where bacterial pollution of the water supply was not indicated by the conventional methods of examination. Authoritative opinion indicates that there is no definite information at present that this physiological derangement is water-borne.[6] Hardness in water has sometimes been held as a cause of various physiological derangements. Authoritative investigators have concluded, however, that where the degree of hardness is such as to make the water fit for domestic use no physiological derangement will result from drinking hard water.[1]

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[1] See also Howard, C. D., "Lead in Drinking Water," *Am. Jour. Pub. Health*, Vol. 13, p. 207, 1923.

[2] See also Smith, Lank, and Smith, *Univ. Ariz. Coll. Agr., Tech. Bull.* 32, June 10, 1931; and Rider, J. H., "Experience with Fluorine in Water," *Jour. A.W.W.A*, November, 1935, p. 1516.

[3] See also Cox, G. J., "New Knowledge of Fluorine in Relation to Dental Caries," *Jour. A.W.W.A.*, November, 1939, p. 1926.

[4] "Evanston to Experiment with Fluorine in Water," *Eng. News-Record*, Apr. 3, 1947, p. 81.

[5] See also Negus, S. S., "Physiological Aspects of Mineral Salts in Public Water Supplies," *Jour. A.W.W.A.*, February, 1938, p. 242.

[6] See also Symposium, ' How Are Public Water Supplies Related to the Incidence of Gastro-enteritis," *Jour. A.W.W.A.*, August, 1937, p. 1137.

457. Cross Connections.[2] To drink the mixture from a recently used water closet, to bathe in the waste liquids discharged from a urinal, to plunge into the diluted contents of a bed pan, to purge a wound with the drainage from an operating table, is worse than disgusting. Such conditions may result from the existence of a cross connection.

A cross connection is any physical connection, either direct or indirect, that will permit or may possibly permit the flow of nonpotable water into a conduit or receptacle containing potable water. A direct connection consists of a continuous conduit leading nonpotable water into the potable water supply. An indirect connection consists of a gap or space across which nonpotable water may fall or be sucked, blown, or otherwise made to enter the potable supply.

Cross connections may occur in the distribution system controlled by the waterworks or in the house plumbing controlled by the consumer. Their effect, however controlled, is to deliver polluted or contaminated water to the consumer resulting in death, disease, or property damage.

The existence of permanent or temporary, active or potential cross connections is more extensive than is generally appreciated. Thousands of cases of water-borne disease resulting from unsuspected cross connections are recorded in the literature. The waterworks may be financially liable and the management may be criminally liable for injuries resulting from the existence of cross connections.

Some common types of cross connections exist where hydrant drains discharge into sewers, water pipes are laid in the same trench with sewer pipes or pass through sewer manholes, outlets to fixtures containing polluted water are above the inlets, etc. Dangerous temporary cross connections occur where a hose attached to a water pipe is allowed to hang in a container of polluted water or which may contain polluted water. Another serious hazard from temporary cross connections occurs along the water front where ships take water from the public water supply, connecting the ships thereto. If the fire pump on the ship, drawing polluted water from the harbor, is started before the shore connection is broken, polluted water is pumped into the public water supply. Ships are known to have polluted themselves, other ships, and shore installations in this manner.

Eternal vigilance is the price of cross-connection prevention. Potential hazards must be eliminated. Single check valves are unsatisfactory safeguards. Under certain conditions double check valves with facilities for drainage between them are permissible. Other protective devices in practice include (1) air-relief valves and vacuum breakers designed to admit air and break any vacuum in the water lines; (2) flush valves and integral vacuum breakers that keep the valves closed when a vacuum is applied; (3) other ingenious types of valves designed to close on actual or impending backflow, none of which is generally accepted by health authorities; (4) swing connections that permit a private water supply to be connected either to the public water supply or to a polluted source of water but not to both simultaneously; (5) nipples, permitting the temporary connection of fire pumps drawing water from a polluted source; (6) the use of open tanks at heights greater than 34 ft.; and (7) the provision of a vertical air gap greater than about one inch, between the inlet and the outlet level of a fixture.

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[1] See also NEGUS, S. S., "Here's to Your Health," *Water Works, Eng.*, March 2, 1938, p. 266.

[2] See also GOUDEY, R. F., "Practical Aspects of Cross Connections," *Jour. A. W. W. A.*, March, 1941, pp. 391 and 495.

Crossovers and interconnections should not be confused with cross connections. A crossover or interconnection is a connection between two potable public water supplies, or between districts in the same supply.

Cross connections are made for various reasons, among which may be included the following: to permit the supplementing of an uncertain or inadequate private water supply by means of the public water supply; to furnish water from or to the public water supply for fire protection; to permit the filling, flushing, or cooling of fixtures or other equipment; and for other reasons. The making of cross connections is difficult to prevent; their existence is hard to detect; and their correction may be inconvenient and expensive. They are best cured by breaking the connection or by abating the conditions that permit the potential danger. Check valves and gate valves are not considered satisfactory safeguards.

CHAPTER XXV

INTERPRETATION OF WATER ANALYSES

458. Purpose of Analysis. Water is analyzed to determine its suitability for various purposes and as an aid in the control of purification processes. It may be analyzed for its sanitary qualities, for its mineral constituents, for the forms of microorganisms in it, for the qualities needed in certain industrial processes, etc. The form of analysis most commonly depended upon to determine the suitability of the water as a public supply is the *sanitary analysis*. Standard methods for the making of sanitary water analyses have been prepared and published by the American Public Health Association.[1] These are widely recognized and serve as a guide for laboratory technicians in making the analysis and to sanitarians in interpreting the results.

The sanitary analysis of a water requires the determination of those characteristics and constituents which might have a bearing upon its suitability as a beverage. A sanitary analysis includes a physical, a chemical, a bacteriological, and occasionally a microscopic examination of the water. Mineral analyses are made primarily to determine the suitability of the water for industrial and other purposes. The constituents and characteristics determined in a sanitary analyses are not always, in themselves, undesirable, but they may point to conditions or qualities that are.

A complete report upon the suitability of a water as a source of a public supply should include both a complete sanitary analysis and a sanitary survey.

459. Sanitary Survey. A sanitary survey of a water is a study of the environmental conditions that may affect its potability. It is not practicable to base an opinion of the sanitary quality of a water upon laboratory analyses alone. It is necessary to have knowledge of the history and environment of the water. The sanitary survey, aided by a complete sanitary analysis, together with knowledge of the significance of the various factors involved, furnishes sufficient data upon which to base the acceptance or rejection of the water as a source for a public supply. Recommendations governing the scope of a sanitary survey have been made by the U.S. Public Health Service.[1] An inspection is often worth more than an analysis and may save both the delay and the expense incident thereto.

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[1] "Standard Methods for the Examination of Water and Sewage," 9th ed., 1946 American Public Health Association.

The scoring of the sanitary quality of water supplies has been attempted by sanitarians, the scoring being based on the sanitary survey and a complete sanitary analysis of the water. The numerical weights given to the various conditions in scoring scales indicate their relative values in the opinions of the respective sanitary authorities. Such scores may be a guide to judgment in the selection of a proposed source for a water supply.[2]

460. Samples. Because of the probable daily and seasonal changes in the quality of a natural water, it is desirable that samples for analysis should be collected frequently and over a long period of time. In the laboratory control of a purification process, samples should be taken with sufficient frequency to permit the necessary changes in the method of operation. The proper collection of a sample of water for analysis is of great importance because of the danger of contamination at the time of sampling or the collection of a nonrepresentative sample. The condemnation of a valuable source of water supply might result from a dirty finger placed in the mouth of the sampling bottle or the collection of a concentration of scum from the surface or sediment from the bottom.

461. Physical Characteristics. The characteristics of water that are reported in a physical analysis include temperature, color, turbidity, and odor.

462. Temperature. The most desirable range of temperatures for a public water supply is between 40 and 50°F. Natural waters are seldom found below 40°. As the temperature rises above 50° the water becomes less palatable and less suited to certain uses, such as air conditioning. Temperatures above 80° are undesirable, and above 90 to 95° the water is unfit for a public supply. Records of the temperature of the raw water are desirable in the operation of a purification plant because the character of the load varies with the temperature, and more coagulant is required by cool than by warm water. Temperature observations at the source in a deep lake or river may guide the operator in securing water from the most desirable depth or location.

463. Color. Color in water has little sanitary significance except possibly to indicate its source. Its presence is undesirable esthetically, it may stain materials with which it comes in contact, and it may complicate coagulation in purification processes. The standards of the U.S. Public Health Service limit the permissible amount of color in an acceptable water to 20 and preferably less than 10 p.p.m.

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[1] *Pub. Health Repts.*, Apr. 10, 1925, p. 693.

[2] CHASE, E. S., before New England Water Works Association, September, 1925; *Eng. and Contr.*, Dec. 9, 1925, p. 1263, HORTON, T., and E. S. CHASE, "Scoring Water Supply Quality," *Eng. News*, Vol 77, p. 134, 1917.

Color in water may be due to mineral or organic matter in solution, as a colloid, or in suspension. It is usually due to organic matter in colloidal suspension. The coloring materials may be procured by surface waters when flowing through swamps or by ground waters by the solution of minerals. Since color can be removed through adsorption by colloids, and since turbidity is caused principally by colloidal matter, turbid waters are less likely to be as highly colored as clear waters subjected to the same conditions. Swampy waters may appear black through the ink formed by a combination of organic acids with tannates and gallates. A yellow tinge may indicate an iron-bearing water colored when the iron is thrown out of solution either by oxidation or through the release of carbon dioxide.

464. Turbidity. Turbidity is a measure of the suspended and colloidal matter in the water. It may be measured by observing the depth at which a standard platinum wire disappears from view when immersed in a sample of water under standard lighting conditions. The color and intensity of the turbidity may reveal, to an experienced eye, the source of the water. For example, a water supply may be taken from a river below a number of tributaries draining different types of drainage areas. The turbidity of the stream would reveal which tributary is contributing most of the flow.

Sediment producing visible turbidity is ordinarily composed of fine particles of sand or clay, and, as such, it is not particularly detrimental to the potability of the water or to most industrial uses. The indication of surface runoff suggests, however, the possible presence of pollution resulting from the washing of the ground surface and may arouse suspicion of the sanitary safety of the water.

Turbidity greater than about 0.5 p.p.m. is noticeable under conditions favorable for observation, as in water held in a white enamel plumbing fixture. Turbidities of surface waters may vary from 1 p.p.m. upward into the thousands. The U.S. Public Health Service[1] standards limit the permissible turbidity to 10 p.p.m. and suggests that, in general, it should not exceed 5.

Variations in the turbidity of a raw water may occur rapidly at a purification plant, resulting in undesirable effects on the purification process and demanding constant vigilance in guarding against them. An "electric eye" has been devised that will warn an operator of changes in turbidity of the water which will require a change in plant operation.[1]

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[1] *Pub Health Repts*, Apr. 10, 1925, p. 693.

465. Odors and Tastes. Odors and tastes in a water supply may result from any one or a combination of such conditions as the presence of (1) microorganisms, either alive or dead; (2) dissolved gases such as hydrogen sulphide, marsh gas, carbon dioxide, or oxygen combined with organic matter; (3) mineral substances such as sodium chloride, iron, carbonates, and sulphates; and (4) phenols and other tarry or oily wastes, especially after chlorination. Some tastes, such as those imparted by oxygen or carbon dioxide, are desirable, but the tastes imparted by most substances are disagreeable. Tastes and odors may originate through the solution of substances as the water flows through or over the ground. They may also originate in the purification procedure as a result of chlorination or through solution of or reaction with substances in the water distribution system such as asphaltic or bituminous coatings on pipes or reservoirs or the zinc in galvanized iron or in brass. An increasing cause of disagreeable tastes in water is the pollution by acid mine wastes and by industrial wastes, of which coal and wood distillation form a large part. These tastes are characteristically medicinal or phenolic, are detectable in extremely dilute concentrations, are intensified by chlorination, and are difficult to remove.

The odor of a water is dependent, to some extent, on its temperature. The odor when the water is hot may be different from what it is when cold. It is usually sufficient in a routine analysis to determine only the odor when cold, but a thorough analysis should include a statement of the odors both hot and cold, since the biological history of the water may be clarified by its odor.

Standard Methods of Water Analysis of the American Public Health Association include a method for both the qualitative and the quantitative determination of odor. No widely accepted method has been devised for the determination of tastes. C. H. Spaulding[2] has suggested a method for the quantitative determination of odor that has received wide recognition. This method requires the determination of the "threshold number" of odor. This is defined as the reciprocal of the highest dilution of the water that can be distinguished from odor-free water. Since the expression of the quality of an odor depends on the experience and training of the observer, the correct expression of odor qualities is difficult. For example, in the qualitative scale of odors suggested by Standard

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[1] "The Radiovisor Turbidity Indicator," *Water and Water Eng.*, July 20, 1933, p. 457. GROSS, D. D., "Electric Eye Guards Water Supply of Denver," *Eng. News-Record*, May 21, 1931, p. 365.

[2] See SPAULDING, C. H., "Quantitative Determinations of Odor in Water," *Am. Jour. Pub. Health*, September, 1931, p. 1038.

Methods[1] appear the terms "earthy" and "peaty." One not acquainted with the odor of peat would probably not describe such an odor correctly.

466. Chemical Analysis. In the sanitary chemical analysis of water a complete determination of the chemical constituents of the water in their quantitative relations is not made. Tests are made that are merely indicative of the quality of the water. The substances and characteristics determined and reported on are, of themselves, harmless in relatively high concentrations. A guide to the interpretation of the significance of these chemical indicators is shown in Table 79.[2]

TABLE 79.—GUIDE TO THE INTERPRETATION AND USE OF SANITARY CHEMICAL ANALYSES OF WATER

(Based on average conditions in Pennsylvania. Results expressed, where possible, in parts per million)

Indicator	Good	Fair	Suspicious	Average sewage (comparative)
Color	0	0 -5	5 and up	High
Odor	None	Negligible	Noticeable	Strong
Turbidity	0 -0.2	0.2 -1.0	1.0 and up	50 and up
Alkalinity	5 -100	5 -125	[1]	50 -150
Hardness	5 -50	50 -100	[2]	
Carbon dioxide	1 -5	5 -10	10 and up	
pH	6.0 -7.2	6.0 -7.5	[3]	5.0 -8.0
Total solids	10 -50	10 -100	100 up	200 -1500
Dissolved	100%-96%[4]	96% -90%	90% and under	75%-25%
Suspended	0 -4%	4 -10%	10% and up	25 -75%
Volatile	0 -10%	10 -20%	20% and up	25 -75%
Fixed	100%-90%	90 -80%	80% and under	25 -75%
Chlorides	1 -2	2 -5	5 and up	20 -150
Oxygen consumed	0 -0.1	0.1 -1.5	1.5 and up	25 -125
Nitrogen:				
Ammonia	0.002-0.02	0.02 -0.05	0.05 and up	5 -50
Albuminoid	0.010-0.05	0.05 -0.1	0.100 and up	7 -35
Nitrites	0.000-0.001	0.001-0.003	0.003 and up	0 -0.2
Nitrates	0.000-0.10	0.10 -0.50	0.50 and up	0 -10.
Iron	0.0 -0.1	0.1 -0.3	0.3 and up	
Manganese	0.0 -0.1	0.1 -0.2	0.2 and up	
Dissolved oxygen	100%	90 -100%	90% and under	0

[1] Below 5 or above 125. [2] Below 5 or above 100. [3] Below 6 or above 7.5. [4] Per cent of total solids.

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[1] "Standard Methods for the Examination of Water and Sewage," 8th ed., 1936, American Public Health Association, with corrections, *Jour. A.W.W.A.*, November, 1939, p. 1945.

[2] See SHANK, J. J., "Chemical Analyses of Water and Their Interpretation," *Penna. Water Works Operators' Rept.*, 1940, p. 63.

467. Oxygen. Oxygen in water may be reported as dissolved oxygen, *oxygen consumed, or biochemical oxygen demand. Dissolved oxygen represents the amount of oxygen in solution.* Natural surface waters of a satisfactory quality should be saturated with dissolved oxygen; a deficiency indicates pollution. Natural underground waters deficient in dissolved oxygen may be satisfactory as a public supply because of the consumption of the oxygen by reactions with underground minerals.

TABLE 80.—SOLUBILITY OF OXYGEN IN WATER
(Under an atmospheric pressure of 760 mm. of mercury, the atmosphere containing 20.9 per cent of oxygen)

Temperature, degrees Centigrade	0	1	2	3	4	5
Oxygen, parts per million	14.66	14.23	13.84	13.48	13.13	12.80
Temperature, degrees Centigrade	6	7	8	9	10	11
Oxygen, parts per million	12.48	12.17	11.87	11.59	11.33	11.08
Temperature, degrees Centigrade	12	13	14	15	16	17
Oxygen, parts per million	10.83	10.60	10.37	10.15	9.95	9.74
Temperature, degrees Centigrade	18	19	20	21	22	23
Oxygen, parts per million	9.54	9.35	9.17	8.99	8.83	8.68
Temperature, degrees Centigrade	24	25	26	27	28	30
Oxygen, parts per million	8.53	8.38	8.22	8.07	7.92	7.63

Organic matter, particularly of a carbonaceous nature, when boiled in acid solution with potassium permanganate will unite with the oxygen in the permanganate. The amount of oxygen removed in this manner from a known quantity of the permanganate is called the *oxygen consumed.* The oxygen-consumed determination is an empirical parameter which makes possible comparisons of the changing quality of any particular water supply, but it is of little value in determining the quality of an unknown source of supply. In general, a water of good quality should be low in oxygen consumed.

Organic life in natural surface waters requires the presence of oxygen. If a sample of the water is placed in a hermetically sealed container, the oxygen dissolved in the original sample will be consumed by the biological metabolism of any organic life that may be present. The amount of oxygen thus consumed is known as the *biochemical oxygen demand,* commonly abbreviated and expressed as B.O.D. It is a measure of the amount of organic life and food in the water. The test is seldom used in the determination of the quality of a water supply, because high contamination is necessary to produce a significant amount of B.O.D.

468. Nitrogen. Nitrogen may be reported in a sanitary water analysis in four different forms: albuminoid ammonia, free ammonia, nitrites, and nitrates. Total organic nitrogen comprises the nitrogen equivalent of nitrogenous compounds in various stages of hydrolysis, from complicated proteins to amino acids. The presence of nitrogen indicates only the presence of organic matter in the water. The state of the nitrogen indicates the relative remoteness of the contact of the water with the organic matter. Either free or albuminoid ammonia is generally considered evidence of pollution unless present in small quantities. The presence of these indicators requires explanation; it proves nothing.

The presence of nitrites and nitrates indicates an organic contact sufficiently remote to permit of some oxidizing action. Nitrate in waters is commonly reported in terms of the nitrogen equivalent, but in mineral analysis results are reported in terms of the acid radical (NO_3) and, as such, in ground waters has no sanitary significance. The quantity of ammonia in natural water is seldom sufficient to require its removal to improve the potability of the water or its quality for other purposes.

The importance of nitrogen determinations in a study of the sanitary quality of a water has been superseded by more direct and useful bacteriological tests.

469. Chlorine and Chlorides. Free chlorine is not found in natural waters, but it may be in waters that have been chlorinated. It is usually desirable that after chlorination a small amount of free chlorine, not to exceed 0.2 p.p.m., shall be present in the water. The existence of the free chlorine indicates that the water has been thoroughly chlorinated. The presence of this amount of free chlorine has no undesirable effect upon the health of the consumer.

Chlorine, as chloride, may be present in natural waters. The presence of chlorides in a concentration higher than is found in natural unpolluted waters in the locality is an indication of pollution by animal excreta or saline industrial wastes. An excess of chlorides is a danger sign because of the nature of their source. Since soluble chlorides are not available, to an appreciable degree, as food for organisms, the concentration of the soluble chlorides can be reduced only by dilution, the change in concentration being a direct measure of the dilution.

470. Alkalinity and Acidity. Alkalinity exists in three forms: as normal carbonate, as bicarbonate, and as hydroxyl or caustic alkalinity. They can be distinguished from each other by titration with methyl orange and phenolphthalein indicators.[1] The three forms of alkalinity can be computed from the pH and total alkalinity by means of equations based on physical chemistry.[2]

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[1] See THEROUX, F. R., and others, "Laboratory Manual for Chemical Analysis of Water and Sewage," p. 140, 3d ed., McGraw-Hill Book Company, Inc., New York, 1943.

[2] See also MOORE, E. W., "Graphical Determination of Carbon Dioxide at the Three Forms of Alkalinity," *Jour. A.W.W.A.*, January, 1939, p. 51.

Although acidity in water is generally considered undesirable, a low acid content is not detrimental to the potability of the water. In fact, pathogenic bacteria die more quickly in acid than in alkaline waters. Some bacteria form spores in acid water. These spores become active when the water is rendered alkaline in purification, thus reducing the apparent efficiency of the process. Acidity may be neutralized by the addition of soda ash in sufficient quantity to furnish a slight excess of alkalinity.

Alkalinity and acidity are expressed in terms of the equivalent weight of calcium carbonate. The terms are thus an expression of the amount or weight present and are no indication of the intensity, or "strength," of the reacting abilities of the substances present. This characteristic is measured by the degree of ionization of these substances in solution.

471. Hydrogen-ion Concentration and pH.[1] Knowledge of the significance of hydrogen-ion concentration is used in waterworks practice principally in the chemical control of purification processes. The expression of the concentration of hydrogen ions in a solution may require so small a figure as to be inconvenient, because of the number of ciphers between the decimal point and the first significant figure. For example, the hydrogen-ion concentration in a molal solution[2] of acetic acid is 0.004, and of a molal solution of ammonium hydroxide is 0.000000000025. In order to avoid the inconvenience of such figures, it has become the custom to express the hydrogen-ion concentration in terms of the symbol pH, which is defined as the logarithm of the reciprocal of the hydrogen-ion concentration. The pH of a molal solution of acetic acid is, therefore, 2.4; and of ammonium hydroxide is 11.6.

It is to be noted that ammonium hydroxide dissociates into NH_4^+ and OH^- ions, apparently containing no H^+ ions, a characteristic of all alkaline electrolytes. Because of the relationship between H^+ ions and OH^- ions in pure water, it is possible to express hydroxyl ions in terms of hydrogen ions.

It is known from the law of mass action that in the solution of an electrolyte the following relation exists:

$$\frac{\text{Concentration of cations} \times \text{concentration of anions}}{\text{Concentration of undissociated molecules}} = \text{a constant}$$

This constant is called the dissociation constant of the electrolytic solution. It has been determined from the electrical conductivity of pure water and from the law of mass action that

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[1] See also CLARK, W. M., "The Determination of Hydrogen Ions," 2d ed., Williams & Wilkins Company, Baltimore, 1923.

[2] A molal solution of a substance contains, in 1 l. of solution, that number of grams of the substance equal to its molecular weight.

$$\frac{\text{Concentration of H ions} \times \text{concentration of OH ions}}{\text{Concentration of undissociated HOH molecules}} = 10^{-14}$$

But since the concentration of H^+ ions must equal the concentration of OH^- ions, for electrical neutrality, and the concentration of undissociated molecules in pure water is practically 100 per cent or unity, the relationship between H^+ and OH^- ions can be written as

$$\text{Concentration of } H^+ = \frac{10^{-14}}{\text{concentration } OH^-} \tag{1}$$

From this relationship the concentration of OH^-, or other negative ions, can be expressed in terms of the equivalent hydrogen-ion concentration.

In pure water the concentration of H^+ ions equals the concentration of OH^- ions for electrical neutrality. Therefore, the hydrogen-ion concentration of pure water is $\sqrt{10^{-14}} = 10^{-7}$, and the pH of pure water is 7.0. All acid substances will have a higher concentration of hydrogen ions, giving a pH below 7.0; and all alkaline substances will have a higher concentration of OH^- or other negative ions, equivalent to a lower concentration of hydrogen ions, resulting in a pH greater than 7.0.

472. Illustrations of pH Computations. In determining the pH of a one-tenth molal solution of acetic acid it has been shown that the dissociation constant is 0.0000171 and that it is 98.7 per cent undissociated. There are, therefore, 0.1 × 0.987 undissociated molecules in the solution. Hence, in a one-tenth molal solution of acetic acid

$$\frac{H^+ \text{ ions} \times C_2H_3O_2^- \text{ ions}}{0.1 \times 0.987} = 0.0000171$$

The concentration of H^+ ions is, therefore,

$$\sqrt{0.0000171 \times 0.0987} = 0.0013$$

The pH of the solution is, therefore, 2.89.

In determining the pH of a molal solution of ammonium hydroxide, a weak base that is practically undissociated, it has been found that the dissociation constant is 0.000018. Hence,

$$\frac{NH_4^+ \text{ ions} \times OH^- \text{ ions}}{NH_4OH \text{ molecules}} = \text{a constant}$$

$$\frac{NH_4^+ \times OH^-}{1.0} = 0.000018$$

and the concentration of OH^- ions is $\sqrt{0.000018} = 0.0042$. The equivalent concentration of H^+ ions, from expression (1), is

$$\text{Concentration } H^+ = \frac{10^{-14}}{0.0042} = 2.5 \times 10^{-11}$$

The pH of the molal solution of ammonium hydroxide is, therefore, 10.6.

Another conception of hydrogen-ion concentration depends on the definition of the normality of a solution. A solution can be said to be normal with respect to hydrogen ions when it contains 1 g. of ionized hydrogen per liter. Since pH is the logarithm of the reciprocal of the hydrogen-ion concentration, it is also the logarithm of the reciprocal of the hydrogen-ion normality. Thus, in a solution in which the hydrogen-ion normality is 0.01, the pH is the logarithm of 1/0.01, or 2.0.

Example: A $0.1N$ solution of hydrochloric acid contains 3.65 g. HCl, of which only 0.1 g. represents ionizable hydrogen. Since HCl of $0.1N$ strength is 84 per cent ionized (at 18°C.), the normality of the solution with respect to ionizable hydrogen is $0.84 \times 0.1N$, or $N/11.90$. The pH of the solution is, therefore, the logarithm of 11.90. or 1.08. A $0.1N$ solution of acetic acid contains the same amount of ionizable hydrogen as does a $0.1N$ hydrochloric acid solution, but it is only 1.3 per cent ionized. The normality, with respect to ionizable hydrogen, is, therefore, $0.013N/10$, or $N/769$. The pH is, therefore, the logarithm of 769, or 2.89.

In an alkaline solution the pH is 14 minus the logarithm of the reciprocal of the hydroxyl-ion concentration. Thus, in a solution in which the hydroxyl-ion concentration is 0.01 the pH is $14 - \log (1/0.01)$, or 12.

Example: A $0.1N$ solution of sodium hydroxide at 25°C. is 92.9 per cent dissociated, and it contains but 0.1 g. of ionizable OH ions. The normality with respect to OH ions is, therefore, 0.929×0.1, or $N/10.76$. The "pOH" is, therefore, the logarithm of 10.76, or 1.03; and the pH is $14 - 1.03$, or 12.97.

Similarly, the apparent degree of dissociation of a $0.1N$ solution of ammonium hydroxide at 25°C. is 1.33 per cent. The normality with respect to OH ions is, therefore, 0.0133×0.1, or $N/752$. The logarithm of 752 is 2.88. The pH is, therefore, $14 - 2.88$ or 11.12.

473. Relation of pH and Acidity. There is no correlation between the expression for the pH of a substance and the expression of its acidity or its alkalinity. The acidity of a solution is the number of parts per million, by weight, of H_2CO_3 or equivalent acid present or as sulphate acidity (SO_4). Alkalinity is reported in terms of the weight in parts per million of CO_3 present in combination with Ca, Mg, K, Na, etc.; and the caustic alkalinity represents the sum total, by weight, of the hydroxides of the same elements. The terms acidity and alkalinity are significant, therefore, in indicating the weight of certain chemical radicals in the solution, these substances having some bearing on industrial, commercial, or physiological properties of the water. Neither indicates the chemical activities of the solution. On the other hand the pH, or hydrogen-ion concentration of a solution is a direct measure of its chemical activities and may also have some bearing on its sanitary and industrial qualities.

Lack of correlation between acidity and pH is indicated by a comparison of these two terms in a 0.1*N* solution of HCl and of H_2SO_4. The acidity of the HCl is twice that of the H_2SO_4, but the pH of the former is 1.08, and the latter 1.2. In this case the substance with the greater acidity has the lower pH. The acidity of a 0.1*N* acetic acid solution is 2,950 p.p.m., and its pH is 2.89.

474. Practical Applications of pH. The pH of natural waters is largely dependent upon the CO_2 equilibriums and lies between 7.0 and 8.0 to 8.5. In general, a lower pH indicates too great an acidity for use as a beverage; but because of the solution of carbon dioxide, the pH of distilled water may be 6.5 or lower. The significance of pH is, therefore, of doubtful value in a sanitary water analysis.

The reduction of the amount of alum required for coagulation has proved the most valuable return from application of knowledge of pH concentration in water purification. Some additional advantages of such knowledge, enumerated by H. P. Eddy,[1] are (1) the prevention of the passage of alum through filters and the prevention of its afterprecipitation in distribution mains; (2) prevention of corrosive action; (3) control of small plant and animal life through ability to control the optimum living conditions, a factor of doubtful value; (4) possible reduction in size of coagulating basins due to better flocculation; and (5) possible increase in the efficiency of bacterial removal. Experiments at Baltimore showed that through the control of pH by the addition of sulphuric acid to the raw water a saving of $1,500 was made in the treatment of 2,200,000,000 gal. of water with alum.

475. Bacteriological Analysis. Interpretations of bacterial analyses are often based upon three commonly used determinations: the 20° (centigrade) count on gelatin, the 37° count on agar, and the *E. coli* index. The bacteria that develop on gelatin at 20°C. may include both harmful and harmless organisms. This count must be considered together with observations of the sanitary survey, in indicating pollution. In general, the gelatin count should not exceed 100 per cubic centimeter, except in some special cases, as in turbid rivers, where it may run as high as 200 per cubic centimeter, provided other conditions are also satisfactory. The bacteria that develop on agar at 37°C. are those which are accustomed to the temperature of the human body. In general their number should not be permitted to exceed 100 per cubic centimeter in a potable water, and it is desirable that it shall be less than 10.

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[1] Eddy, H. P., "The Significance of Hydrogen Ion Concentration in Water Purification," *Jour. New Engl. Water Works Assoc.*, Vol. 35, p. 385, 1921.

476. The E. Coli Index. The *E. coli* index[1] for a single sample is the reciprocal of the smallest quantity of the sample, expressed in cubic centimeters, which would give a positive *E. coli* test. For example, if three dilutions of a sample, 1:1, 1:10, and 1:100, are analyzed and a positive result is found in both of the higher dilutions, the smallest quantity of the sample that will give a positive result is one one-hundredth of a cubic centimeter. By definition the *E. coli* index is, therefore, 100. When an apparently anomalous but, nevertheless, rationally probable case occurs wherein a negative result is obtained in a dilution between two positive results, the positive observation at the higher dilution is interchanged in the record with its next lowest negative result and the index is based on the changed data. The index is more accurate when computed from a series of tests. Under such conditions the procedure in computing it is as follows:

1. Record the ratio of positive to total tests, arranged in the order of magnitude of the quantities examined.
2. Record the differences between successive ratios.
3. Find the product of each difference and the reciprocal of the larger quantity used in computing the ratio from which the difference was taken.
4. The *E. coli* index is the sum of these products.

Example: Daily *E. coli* determinations were made for a month at a particular filter plant, with the following results:

(1) Quantities tested, cc.	(2) Number tests	(3) Number positive	(4) Ratio (3):(2)	(5) Differences between lines in col. (4)	(6) Product
1.0	30	20	0.67		
0.1	30	5	0.167	0.503	0.503
0.01	30	4	0.133	0.034	0.34
0.001	20	1	0.05	0.083	8.3
E. coli index					9.13

The reciprocal method of determining the *E. coli* index is not altogether satisfactory because of anomalies that may arise in its use. In the reciprocal method, observations in column 3 must fall on the probability curve to give consistent values in column 6. For example, if in line 4, column 3, the figure were 2, the value in column 6 would be 3.3. Obviously, for a greater number of positive tests we should have a higher index. The error results from assuming a value of 2, which falls off the probability curve in column 3. The "most probable number" method has been standardized.[1,2]

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[1] Designated as the "indicated number" in "Standard Methods for the Examination of Water and Sewage," p. 205, 9th. ed., 1946, American Public Health Association.

477. The Most Probable Number (M.P.N.) of E. Coli. Because of the anomalies that may develop in determining the *E. coli* index, some analysts consider that the "most probable number" of *E. coli* is of greater precision and value.[3] The general form of the probability curve is, in this case,

$$y = \frac{1}{a}[(1 - e^{-N_1\lambda})^p(e^{-N_1\lambda})^q][(1 - e^{-N_2\lambda})^r(e^{-N_2\lambda})^s][(1 - e^{-N_3\lambda})^t(e^{-N_3\lambda})^u][\;\cdot\;\cdot\;\cdot\;]$$

where N_1, N_2, N_3, = sizes, in cubic centimeter, of portions examined
p, r, t, = numbers of portions of respective sizes giving positive tests for *E. coli*
q, s, u, = numbers of portions of corresponding sizes giving negative tests for *E. coli*
λ = concentration of *E. coli* per cubic centimeter
y = probability of occurrence of the particular results if the concentration from which the result was drawn was λ
e = base of Napierian logarithms (2.72)
a = a constant for any particular set of observations the determination of the value of which may, therefore, be omitted in routine computations of λ

For example, in a series of observations the results of the *E. coli* determinations gave

10 cc.	10 cc.	10 cc.	10 cc.	10 cc.	1 cc.	0.1 cc.
+	+	+	−	−	+	−

In this case, then,

$$N_1 = 10, \quad N_2 = 1, \quad N_3 = 0.1, \quad p = 3, \quad r = 1,$$
$$t = 0, \quad q = 2, \quad s = 0, \quad u = 1$$

Substituting and transposing,

$$ay = (1 - e^{-10\lambda})^3(1 - e^{-\lambda})(e^{-10\lambda})^2(e^{-0.1\lambda})$$

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[1] "Standard Methods for The Examination of Water and Sewage," 8th ed., pp. 218 and 223, American Public Health Association, 1936.

[2] See also POMEROY, R., *Jour. A.W.W.A.*, March, 1940, p. 478.

[3] See also HOSKINS, J. K., "The Most Probable Numbers of *B. Coli* in Water Analysis," *9th Ann. Rept. State Sanitary Eng.*, p. 61, 1928.

The most probable number of *E. coli* for the series is determined by finding the value of λ that will make y a maximum. In the particular illustration given above, this occurs when $\lambda = 0.12$. Hence, in this case, the most probable number of *B. coli* is 12 per 100 cc.

478. Microscopical Analysis. The microscopical examination of water may serve any one or more of the following purposes:[1] (1) to explain the cause of color and turbidity and the presence of objectionable odors and tastes in water and to indicate methods for their removal, (2) to aid in the interpretation of chemical analyses, (3) to identify the source of a water that is mixing with another, (4) to explain the clogging of pipes and filters and to aid in the design and operation of waterworks, (5) to indicate pollution by sewage or industrial wastes, (6) to indicate the progress of the self-purification of streams and other bodies of water, (7) to aid in explaining the mechanism of biological sewage-treatment methods, and (8) to aid in the study of the food of fish, shellfish, and other aquatic organisms.

Since both the horizontal and the vertical distribution of microorganisms is uneven in most bodies of water, whether lakes or running streams, the selection of sampling points must be made with care to ascertain the most desirable points for the procurement of representative samples. Factors to be considered include the location of tributary cultivated land, whether shallow or deep water is near by, adjacent swamp areas, the proximity of tributaries, etc. The application to the entire body of water of the findings in a sample taken from a shallow area of a deep reservoir would obviously be incorrect and misleading. It is advisable that the microscopical examination be made quickly after the taking of the sample because of the relatively rapid changes in the character of the more fragile and delicate of the living organisms.

In the control, by microscopical examination, of the quality of water in a reservoir or other source of water supply, samples should be taken, at least weekly, at desirably located sampling points.[2] Indications of approaching changes in biological life may be observed by the trend of lines plotted to show the numbers of various species at each sampling point and period. In this way steps may be taken in time to safeguard the quality of the water. Reports of microscopical analyses should include statements of the kind of organisms present, their approximate concentration, and the standard units present.

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[1] "Standard Methods for the Analysis of Water and Sewage," p 176, 8th ed., 1936, and 9th ed. 1946, American Public Health Association.

[2] See also PURDY, W. C., "Methods for the Biological Examination of Water," *Proc. 9th Ann. Water Works School, Univ. Kansas*, 1931, pp. 21 and 31.

479. Mineral Analysis. Mineral analyses are made to determine the industrial and commercial qualities and possibilities of water. They are more expensive to make than sanitary analyses, and they are not considered of value in interpreting the sanitary quality of the water.

In the performance of a mineral analysis it is customary to make only the determinations shown in the first column of Table 81. The substances that these ions are supposed to form are assumed by combining them in accordance with some rule of hypothetical combination.[1] Standard Methods[2] specifies that hypothetical combinations may be reported by combining the milliequivalents of basic and acid radicals according to the following scheme of uniformity:

1. Silica, as found, except in alkaline waters containing silicates.
2. Iron and aluminum oxides, as found, except in special cases of acidity.
3. Combinations of the positive radicals should be made with the negative radicals in the following order:

Positive Radicals	Negative Radicals
Calcium	Carbonate
Magnesium	Hydroxide
Sodium	Sulphate
Potassium	Nitrate
	Chloride

Hypothetical combinations for a typical ground water, computed by the above method, are shown in column 4 of Table 81.

A study of these combinations may reveal data useful in reaching a conclusion as to the value of the water.

480. Industrial Water Analysis. Methods for reporting the analyses of waters for industrial uses differ from procedures used in reporting either sanitary analyses or mineral analyses. Among the major differences noted in the tentative standard methods of the American Society for Testing Materials[3] may be included constituents reported and units used. Constituents reported include, for example: total CO_2, metallic oxides, silica, the important metallic cations such as Ca^{++}, Mg^{++}, Na^{+}, K^{+}, Fe^{++}, Fe^{+++}, Al^{+++}, Mn^{++}, Mn^{+++} and nonmetallic anions such as CO_3^{--}, HCO_3^{-}, OH^{-}, PO_4^{---}, HPO_4^{--}, $H_2PO_4^{-}$, SO_4^{--}, HSO_4^{-}, Cl^{-}. Units used in reporting concentrations are entitled "equivalents per million," or e.p.m. An equivalent per million is a gram equivalent weight per million grams of the solution. It can be converted to parts per million by multiplying it by the equivalent weight of the ion. For example, 2.2 e.p.m. of HCO_3^-, whose equivalent weight is 61, equals 134 p.p.m. The use of equivalents per million is advantageous in stoichiometrical computations.

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[1] See also LANGELIER, W. F., and H. F. LUDWIG, "Graphical Methods for Indicating the Mineral Character of Natural Waters," *Jour. A.W.W.A.*, March, 1942, p. 335.

[2] "Standard Methods for the Examination of Water and Sewage," p. 54, 8th. ed., 1936, and p. 7, 9th ed., 1946, American Public Health Association.

[3] See "Tentative Method of Reporting Results of Analysis of Industrial Waters," *A.S.T.M. Standards Supp., Part* III, p. 541, 1940.

TABLE 81.—MINERAL ANALYSIS OF A SAMPLE OF GROUND WATER SHOWING HYPOTHETICAL COMBINATIONS

Determination made	P.p.m.	Hypothetical combination	P.p.m.
Iron, Fe	2.1	Silica	80
Manganese, Mn	....	Iron oxide, Fe_2O_3	3.9
Silica, SiO_2	80	Aluminum oxide, Al_2O_3	27.0
Alumina, Al_2O_3	27	Calcium carbonate, $CaCO_3$	72.0
Calcium, Ca	1,220	Calcium hydroxide, $Ca(OH)_2$	
Magnesium, Mg	530	Calcium sulphate, $CaSO_4$	45.8
Ammonia, NH_4	13.8	Calcium nitrite, $CaNO_2$	2.4
Sodium, Na	5,297.2	Calcium nitrate, $CaNO_3$	1.3
Potassium, K	627.3	Calcium chloride, $CaCl_2$	3,260
Sulphate, SO_4	32.3	Magnesium chloride, $MgCl_2$	2,080
Nitrate, NO_3	0.8	Sodium chloride, NaCl	13,480
Nitrite, NO_2	1.3	Potassium chloride, KCl	1,198
Chloride, Cl	12,250	Ammonium chloride, NH_4Cl	41.1
Alkalinity, M.O.	72		
Nonvolatile	28.2		
Residue	32,100		

481. Colloids. Colloids may affect the quality of raw water, action in coagulation, and the process of filtration. Knowledge of their characteristics is of importance in the procurement and purification of public water supplies.

Finely divided matter may exist as a true solution, as a colloidal suspension, or as a true suspension. A true solution is composed of particles of the size of molecules, and it is supposed that the molecules of the dissolved substance and the solvent are intimately mixed, the smaller molecules of the substance being interspersed between the larger molecules of the solvent. The particles in a solution grade in size to become particles in a "colloidal solution," or sol, and thence to sizes in a suspension. The smaller particles in a sol, which can pass from one point to another between the larger particles, are called the continuous, or closed, phase or, frequently, the dispersion medium, and the larger particles constitute the disperse phase. For example, smoke represents a colloidal state, or sol, in which the atmosphere is the dispersion medium and the fine particles of carbon in the smoke are the disperse phase. Activated carbon may be considered as an example of a sol in which the dispersion medium is the gas and the disperse phase is the carbon.

Colloidal particles are assumed to have dimensions that do not exceed 1μ, or micron, and that may approach the size of a molecule, *i.e.*, 0.1 to 0.5 millimicron. The existence of such particles can be detected by the Tyndall phenomenon and the Brownian movement. The presence of colloidal particles can be detected by the Tyndall cone and by the fact that they affect very slightly, if at all, the boiling and freezing points of the dispersion medium. Colloids diffuse slowly, do not pass through parchment paper, and do not settle.

A better understanding of the reason for the special reactions of colloids may be gained through a conception of the specific surface of a colloid, *i.e.*, the ratio of its surface to its volume. If a cubic centimeter could be divided into the smallest colloidal particles, each a cube with an edge of 0.1 millimicron, the total surface area would be 600,000,000 sq. cm., or 15 acres, per cubic centimeter of material. With such an enormous surface in contact with the continuous phase it is to be expected that characteristic reactions will take place in sols (suspensoids).

A suspensoid, or sol, in which water is the dispersion medium and particles of clay, silica, or precipitated chemicals are the disperse phase is the most common form of colloid encountered in water purification. Suspensoids may be precipitated by the addition of an electrolyte, by changing the concentration of the disperse phase, or by means of electricity. The electrical charges on the colloids are apparently neutralized by those of the electrolyte, permitting the colloidal particles to come together. The colloidal particles in a suspensoid are sometimes protected against the action of precipitating agents by the presence of an external agent. Such particles are called protected colloids. For example, gelatin added to a silica sol will make the precipitation of the sol more difficult than if the gelatin were not present.

An emulsoid differs somewhat from a suspensoid in that the disperse phase combines, to some extent, with the continuous phase, rendering the breaking of the emulsion and the precipitation of the disperse phase difficult. Soap, gelatin, some oils, and many sludges found in water and sewage treatment mix with water to form emulsoids. Emulsoids of amphoteric substances, such as gelatin and other proteins, may be precipitated by the adjustment of the pH or the addition of certain soluble salts. Emulsoids may be broken by agitation, by altering the viscosity, or by changing the surface tension of the emulsifying or stabilizing agent. A change of temperature may change the viscosity, or various chemicals or electricity may be used to change the viscosity or the surface tension or both.

CHAPTER XXVI

PLAIN SEDIMENTATION

482. Process. The process of treating water by plain sedimentation requires the retention of the water in a basin so that the suspended particles may settle as a result of the action of gravity and other forces. Plain sedimentation is suitable as a purification process only for relatively pure waters containing undesirable amounts of suspended solids. There is no upper limit to the amount of settleable solids that may be present in water to be treated by plain sedimentation, as experience has shown that water containing a high concentration of suspended matter is more easily clarified by sedimentation than a water containing a low concentration of suspended matter, particularly if the particles are finely divided without being colloidal. Plain sedimentation may be useful as a process preliminary to further treatment in order to lighten the load on subsequent processes. Better operation of subsequent purification processes is secured through the less variable quality of the water delivered to them; there is a lowered cost in the cleaning of the chemical coagulating basins; no chemical is lost with the sludge discharged from the plain settling basin; and fewer chemicals are used in subsequent treatment. All surface waters are not sufficiently turbid to justify presedimentation.

Sedimentation basins are now operated almost exclusively on the continuous-flow principle, in which water flows through the basin continuously, entering and leaving at the same rate. The fill-and-draw method of operation is obsolete. Sludge may be removed from the basins by emptying the basin and flushing or scraping the sludge from it, or it may be removed intermittently or continuously by mechanical devices without interrupting the operation of the basin. The use of continuous mechanical cleaning is becoming more common, as the extra cost of the mechanisms can be saved through the reduction of the size of the basins required to give the same or better results.

Some results of plain sedimentation are shown in Table 82.

It is evident from these that treatment by plain sedimentation alone cannot serve to produce a water safe as a public supply.

483. Sedimentation-basin Design. Among the forces and conditions affecting the sedimentation of matter suspended in water are the following:

The force of gravity.

The size and specific gravity of the settling particles.

Coagulation or coalescence of the settling particles.

The depth and shape of the basin.

The viscosity (temperature) of the water.

The presence of convection and other currents.

The method of operation of the basin.

Electrical phenomena.

Biological activities.

TABLE 82.—REDUCTION OF TURBIDITY AND BACTERIA IN VARIOUS STAGES OF WATER TREATMENT

	Toledo,[1] Ohio, August, 1923, to July, 1924	Youngstown,[1] Ohio, October, 1923, to September, 1924	Niagara Falls,[1] N.Y., October, 1923, to September, 1924	Cincinnati,[1] Ohio	Louisville,[1] Ky.
Turbidity:					
Raw water	185	56	21		
Per cent removed by plain sedimentation[2]	......	82.7			
Per cent removed by coagulation	87.0	92.5	42.2		
Bacteria, 48 hr. at 20°					
Raw water, 1,000 per cubic centimeter	14.1	8.69	9.96		
Per cent removed by:					
Plain sedimentation					
Coagulation	80.0	0.32[3]	50.1		
Filtration	94.2	0.022[3]	85.6		
Chlorination	96.9	0.053[3]	99.86		
B. coli index:					
Raw water	1,650	5,450	8,290	0.685	30.9
Per cent removed by:					
Plain sedimentation	....	....	....	47.2	92.9
Coagulation	81.9	0.005[3]	64.9	87.6	94.8
Filtration	98.8	0.007[3]	0.98[3]	98.5	0.097[3]
Chlorination	0.27[3]	0.001[3]	0.015[3]	0.355[3]	0.0193[3]

[1] "Studies of the Effect of Water Purification Processes," *Pub. Health Service, Bull.* 172, 1927

[2] Percentage removal shown is the percentage of the amount in the raw water.

[3] 100 minus the percentage removal.

Other forces and conditions probably exist but are neither definitely nor easily recognizable. The difficulties of making allowances for all of these forces and conditions is apparent, with the result that the design of plain sedimentation basins is usually based on experience with other waters of a similar character or upon tests made upon the particular water to be treated. A committee of the American Society of Civil Engineers states:[1]

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[1] "Water Treatment Plant Design," *Manual of Practice* 19, 1941, p. 43. American Society Civil Engineers.

> . . . *despite the various influencing factors* . . . best results will be obtained with a straight flow through the basin, where the [coagulated] water is introduced at one end through a diffuser wall, passed through enough training walls along its line of travel to "iron out" discrepancies in velocity of flow, and taken out at the other end over a submerged weir or enough large openings so as not to influence the uniform velocity through the basin. If a uniform velocity can be maintained throughout the length of basin, conditions for subsidence become ideal, and the ideal is approached in long, narrow basins such as tunnels, large pipe lines, and canals.

Important conditions to be considered in the design of a rectangular sedimentation basin, either with or without interruption of operation for the removal of sludge, include (1) the period of retention, (2) the number and capacity of basins, (3) the velocity of flow, (4) the depth of the basin, (5) the relation of the length to the width of the basin, (6) the inlet and outlet devices, (7) the cover or roof, (8) the provisions for cleaning, and (9) the sludge storage capacity. Many of these conditions depend on others, on the quality of the water to be treated, and on the desired results.

Materials used in the construction of settling basins, filters, and other containers are the same as those used for reservoirs, as described in Chap. XXI.

484. Theory and Practice. It takes time for a settling particle to reach the bottom of a sedimentation basin. This time is provided by retarding the velocity of flow and making the length of flow, or the period of retention, and the depth such that the settling particle will reach the bottom before reaching the outlet. It would seem that the shorter the period of retention, the shorter the required length of basin and the less the required depth. The absurd conclusion is reached that a basin without length or depth is desirable. Hence, experience in the design of basins must be depended on in design. Attempts have been made to formulate mathematical expressions that would be applicable to the determination of the dimensions of sedimentation basins. Notable among these attempts are those of Hazen,[1] Slade,[2] Camp,[3] and Carpenter and Speiden.[4] In view of the large number of conditions involved and the unknown "constants" involved, it continues to be customary to base the design of sedimentation basins on experience.

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[1] Hazen, Allen, "On Sedimentation," *Trans. A.S.C.E.*, Vol. 73, p. 43, 1904.

[2] Slade, J. J., Jr., "Sedimentation in Quiescent and Turbulent Basins," *Trans. A.S.C.E.*, Vol. 102, p. 289, 1937.

[3] Camp, T. R., "A Study of the Rational Design of Settling Tanks," *Sewage Works Jour.*, September, 1936, p. 742.

[4] Carpenter, L. V., and H. W. Speiden, "Notes on Sedimentation," *Sewage Works Jour.*, March, 1935, p. 200.

485. Depth. Depths of horizontal-flow sedimentation basins of all types range normally between 10 and 18 ft., with 10 to 15 ft. being most common. These depths include allowance for sludge storage, if provided. Greater depths, up to 20 to 25 ft., are found in vertical-flow basins.

486. Period of Detention. The term "period of detention" as used in practice, is equal to the volume of the basin divided by the volumetric rate of flow through the basin. It is equal to the time required to fill the basin at the given rate of flow. This period is greater than the "flowing-through period," which is the average time required for a small volume of liquid to pass through the basin at the given rate of flow. The efficiency of displacement is the ratio of the flowing-through period to the period of detention, multiplied by 100. Efficiencies of basins now in use vary roughly between 5 and 50 per cent. Periods of detention found in existing basins vary from a portion of an hour to many days, but, since the greater removal occurs during the first few hours, basins are usually designed for a period to be counted in hours rather than in days. If the basin is to be well designed, a test should be made to determine the proper period of detention. Results of such studies on some turbid river waters are shown in Fig. 221. The shortest detention periods used, between 1½ and 3 hr., are to be found in mechanically cleaned basins.

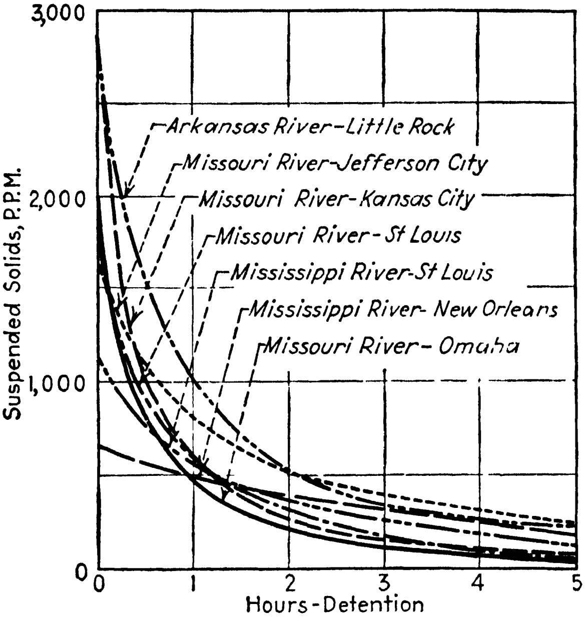

FIG. 221.—Results of sedimentation studies in turbid river waters. (*From Jour. A.W.W.A., Vol* 19, *p.* 281, 1928.)

487. Capacity and Number of Basins. The volume of flow through the plant per unit of time multiplied by the assumed period of retention, in the same units of time, will give the net capacities of all the basins to be provided in a plant. In large plants an excess capacity may be provided to allow for unexpected situations, but in small plants allowance of extra capacity is not always necessary, as shutdowns of the basins can be timed for conditions when the quality of the raw water is such as to permit a shorter period of sedimentation.

It is desirable to design a plant to permit the use of different periods of retention in operation to conform to variations in the quality of the raw water. This can be done by providing a number of units whose combined capacities are equal to the total required capacity for the plant. Where two or more basins are provided, it may be desirable to make them of different capacities so that a greater number of different periods of retention may be secured. For example, if a plant treating 8,000,000 gal. per day is provided with two basins, one with a capacity of 1,000,000 gal. and the other of 2,000,000 gal., the possible periods of retention are 3, 6, and 9 hr. Flexibility in operation can be obtained by connecting the basins in such a manner that any basin can be operated alone or in any combination with the other basins, either in parallel or in series. The number of units to be built is limited by the cost and the desired permissible fluctuations in retention period. Under conditions in which treatment cannot be interrupted there should never be less than two units, preferably three, to permit the shutdown of one basin.

488. Circular Settling Basins. Circular sedimentation basins are less commonly used than rectangular basins in water purification because of economical considerations. However, in small plants, and where steel can be used economically, circular tanks may be used, as their hydraulic characteristics, when properly designed, may be made satisfactory.[1] In general, these tanks are designed to admit the influent at the center of the tank and to withdraw the effluent into a peripheral trough at the surface. Some essential considerations to successful design include (1) the creation of a uniform velocity in the vertical riser feeding the tanks; (2) a satisfactory diffuser to distribute the water evenly without undue loss of head; (3) the absence of turbulent velocities within the tank; (4) the provision of a zone of velocity less than 10 f.p.m. and preferably below 1 f.p.m. at the tank bottom adjacent to the sludge blanket; and (4) protection from wind disturbance.

489. Velocity of Flow. The nominal velocity of flow in a plain sedimentation basin is equal to the volumetric rate of flow divided by the vertical cross-sectional area of the basin. In general, the slower the velocity, the more efficient the removal of settling solids, other things being equal. No sedimentation basin should provide a velocity greater than 1 f.p.m., and many basins are in existence with velocities of $^{1}/_{10}$ to $^{1}/_{100}$ f p.m.

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[1] See also HUBBELL, G. E., "Hydraulic Characteristics of Various Circular Settling Tanks," *Jour. A.W.W.A.*, Vol 30, p 335, 1938.

490. Surface Area. Practice has shown that the ratio of surface area to volume of tank is a factor to be considered. It is recommended that the load per square foot of surface area of settling basin shall be between 300 and 4,000 gal. per day for granular solids, 800 to 2,000 gal. per day for amorphous slow-settling solids, and 1,000 to 1,200 gal. per day for flocculent material. Too great a surface area in relation to tank volume is conducive to the creation of currents by the wind.

491. Relation of Length to Width. A long, narrow basin is preferable, from the viewpoint of sedimentation efficiency, to one so wide that cross currents may develop. A square is the best shape for a single rectangular basin to give the least cost of construction. If a large number of rectangular basins are to be built with a common wall between each pair of basins, a length somewhat greater than the width is desirable for economy. If the unit costs of the various parts of the basins are known, the dimensions for economy can be determined by the calculus. In practice it has been found desirable to limit the minimum ratio of length to width of flowing-through channel to 4:1, with a maximum channel width of about 40 ft.

Where conditions of economy or convenience require a ratio of length to width of basin smaller than 4:1, a larger ratio of length to width of flowing-through channel can be obtained by the provision of round-the-end baffles, as illustrated in Fig. 222. In order to minimize eddy currents, the distance between the end of the baffle and the end wall of the basin should be equal or exceed the width of the flowing-through channel.

492. Baffles. Baffles are sometimes used to improve the distribution of velocities and to avoid dead spaces. Their effect should be understood before they are installed. Guesswork installations may make conditions worse. Improvement may be expected[1] where the design is based on knowledge derived from test or experience.

493. Inlet and Outlet Devices. The inlet and outlet devices in basins with horizontal flow should be arranged to distribute and collect incoming and outgoing water as uniformly as possible across the vertical cross section of the flowing-through channel.

Such devices include a sharp-crested weir, an overflow trough similar to the weir, a manifold with two or more openings into the flowing-through channel, and a perforated baffle at the end of the flowing-through channel.[1] The choice will depend on the cost and the results desired. The perforated baffle is probably the most effective and the most expensive. Velocities through the diffuser-wall slots or perforations of 0.4 to 0.8 f.p.s., which require a head of 0.01 to 0.02 ft., have proved satisfactory in producing uniform flow. The sharp-crested weir is less expensive, and it can be so constructed as to permit the adjustment of its crest elevation.

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[1] See also HOPKINS, E. S., "Baffled Basins Improve Sedimentation," *Eng. News-Record*, Apr. 3, 1947, p. 80.

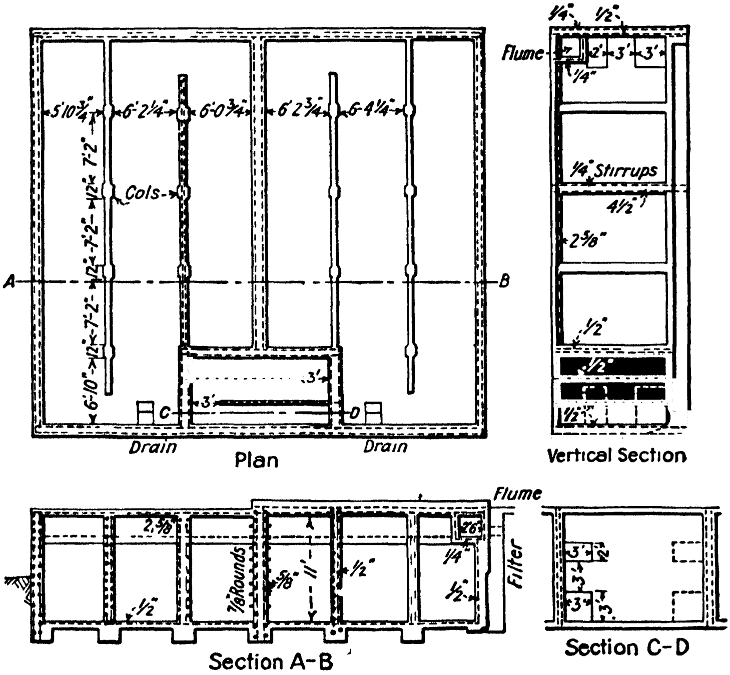

FIG. 222.—Waterworks settling basin at Ironwood, Mich. (*From Eng. News, Vol.* 70, *p.* 1193.)

In general, it is less effective to attempt to control the currents in a sedimentation basin by means of the outlet devices than by the inlet devices. The perforated baffle wall is, therefore, seldom used as an outlet device except in basins designed to permit reversal of flow.

Where the water entering the basin is colder than that already in the basin, a current is created along the bottom which may reduce the efficiency of the results. The installation of low overflow baffles, which form pockets for the collection of the sludge, partially protecting it from the bottom current, may be helpful in reducing the condition. The reverse situation, in which a warm current enters the basin that contains colder water, results in the formation of a surface current across the basin. The results, under such circumstances, are not so undesirable, because sedimentation is occurring in the equivalent of a very shallow basin and the bottom sludge is undisturbed. Surface currents can be broken up to some extent by submerging the inlet or by use of hanging baffles with their lower edges submerged 12 to 18 in. below the surface.

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[1] See also MORRILL, A. B., "Sedimentation Basin Research and Design," *Jour. A.W.W.A.*, 1932, p. 1442.

Outlets from sedimentation basins may be designed as overflow weirs or troughs, sometimes called launders. The location and the length of these launders in relation to the flow through the basin affect the performance of the basin. In primary sedimentation basins to remove settleable solids only, the outlet may be placed at the end of a rectangular basin opposite the inlet or on the periphery of a circular tank with the inlet at the center. Where flocculent material is to be removed, the launder may be placed somewhere between the inlet and the wall of the basin, to avoid the effect of "density currents" that tend to carry settled solids to the extreme end of sedimentation basins.

To minimize surges in a sedimentation basin the rate of flow per foot of the outlet weir should not exceed 200,000 gal. per day and should preferably be less than one fourth of this. In some cases it may be desirable to make an appreciable portion, if not the full length of the weir, vertically adjustable in order to place better control of the basin in the hands of the operator.

494. Covers. It is best, where possible, to leave plain sedimentation basins uncovered, as the conditions of operation can be better observed. The principal purposes of covers may be to avoid difficulties from wind and ice, to prevent contamination of the water, to avoid the growth of organisms, and to improve the appearance of the plant. Where covers are used, care must be taken to provide access to the basin, to permit the ingress and egress of air to compensate for fluctuations of the volume of water in the basin, to avoid fungus growths, and to avoid egg laying by flying insects.

495. Sludge Storage. The allowance made for the storage of sludge depends on the period between cleanings and the amount of material removed by the water. Only test and experience can dictate the proper allowance. Since the proper depth of a sedimentation basin is indefinite, designers decide on a convenient depth and assume that a portion of the capacity of the basin will be required for sludge storage.

The quantity of sludge to be handled may be roughly approximated from the expression

$$Q = \frac{SP}{240P_1} \tag{1}$$

where Q = tons of wet sludge to be handled per million gallons of water treated

S = suspended matter in raw water, p.p.m.

P = percentage of suspended solids removed by plain sedimentation

P_1 = percentage of solids in the sludge

Where continuous mechanical removal of the sludge is provided, no sludge storage space is needed.

496. Draining and Cleaning. The cleaning of the basin may be provided for by making the bottom a series of steep-sided hoppers or by sloping the bottom slightly to a gutter or to an outlet through the wall. Under such conditions the bottom of the basin should slope not less than 2 to 100. The outlet should be controlled by a sluice gate or valve, as shown in Fig. 223; or the outlets may be placed in the bottom of the basin and protected by mud valves; or the bottom of the basin may be open to the drain as shown in Fig. 224. Control is provided by a single valve placed beyond the walls of the settling basin. It is undesirable to place an underdrain beneath the floor or walls of a settling basin when there is to be water under pressure in the drain because of the difficulty in cleaning the drain or making repairs. Leaks in such a drain may undermine the floor or the wall of the basin.

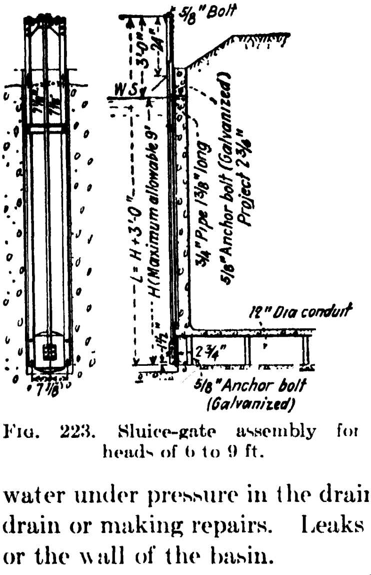

FIG. 223. Sluice-gate assembly for heads of 6 to 9 ft.

If a drainage valve is opened when the basin contains a quantity of water, the force of the current will remove some of the settled solids. The remainder must be removed by hand or by machine or by hydraulic flushing. It is desirable to provide water pipes for this purpose which will furnish a quantity of water under pressure to a flushing hose and nozzle.

497. Continuous Sludge Removal. In some sedimentation plants recently constructed, principally in connection with chemical coagulation or softening, provision has been made for the continuous removal of sludge, obviating the need for allowing sludge storage space in the design of the basin and avoiding the necessity for periodic shutdowns to clean the basin. Continuous cleaning is accomplished by a series of wood or metal scrapers, or squeegees, which are dragged along or around on the bottom of the settling basin, sweeping the deposited material to one end, or to the center, of the basin. The linear velocity of the squeegees does not exceed about 15 f.p.m. The slow velocity does not create undesirable currents and requires but little power for operation. In one instance 2.8 hp. was required to drive a 200 ft. diameter cleaner.

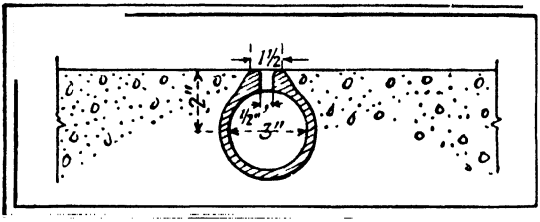

FIG. 224. Detail of underdrain and orifice for sludge-removal system, Moorestown, N.J. deferrization plant. (*From Proc. Am. Soc. Mun. Imp.*, 1916.)

In some installations, experience has indicated that it is more economical to install sedimentation basins with continuous sludge removal because of the saving in sludge storage capacity, the increased efficiency of the results obtained, and the avoidance of sludge decomposition. The continuous return, to the inlet of the sedimentation basin, of a portion of the deposited sludge has been found, under proper conditions, to increase the efficiency of the process.

CHAPTER XXVII

COAGULATION

498. Sedimentation with Coagulation. It has been found that by the addition of certain chemicals to water an insoluble, gelatinous, flocculent precipitate will be formed which, in its formation and descent through the water, will adsorb and entrain suspended and colloidal matter, will hasten its sedimentation, and will remove particles more completely and rapidly than by plain sedimentation. The addition of a coagulant may serve also to aid in the removal of color, odor, and taste from water. The chemicals ordinarily used as coagulants, when properly applied, are harmless to the consumer of the water.

In practice the method consists of mixing the coagulating chemicals with the water in sufficient strength to produce the required precipitate and then allowing the water to stand quietly in, or to flow slowly through, a sedimentation basin in which the coagulant settles to the bottom.

The period of retention to be used can be determined best by test made under operating conditions. The quality of the water, the kind of coagulant, and the physical conditions in the coagulating basin all affect the necessary period of retention. The desirable period of retention at any plant will vary with the quality of the water, the temperature, and other local conditions. Normal variations at different plants are found between 1 and 12 hr. Wider variations have been recorded. It is desirable, therefore, to design coagulating basins so that the period of retention may be varied.

Sedimentation with coagulation is not commonly used as a complete method of treatment but is used rather as a preliminary step to prepare water for filtration. Some results of coagulation are shown in Table 82.

499. Control of Coagulation. The proper amount of coagulation, the method of mixing, the method of flocculation, the period of coagulation, the operation of the coagulating basin, and other operating conditions can be determined only through experience in the laboratory and the field with the water to be treated. The chemical control of coagulation is made difficult owing to the extremely dilute solutions in which reactions must take place. Among the characteristics of the water and the conditions of operation that affect the control of coagulation are color, turbidity, temperature, alkalinity, pH, hardness, and free carbon dioxide. It is desirable that experience with these factors be plotted against the coagulant dose and treatment results, so that repetition of the conditions of treatment may permit the finding of equal or better results in operation.

Temperature is an important factor in the control of coagulation, experience showing that, in general, floc forms more quickly at relatively low temperatures and that larger amounts of coagulant are required in summer than in winter. This apparent anomaly may possibly be explained by the fact that coagulation is a manifestation of electrochemical phenomena and that the seasonal adjustments of pH are more effective than those of temperature in affecting it. At temperatures below 40°F., difficulty may be encountered in securing good floc formation.

Since floc formation is closely related to hydrogen-ion concentration, the control of pH is an important factor in the control of coagulation. The point of optimum coagulation may be as low as pH 3.8 for highly colored soft swamp waters, or it may be greater than 8.0 with other waters. The optimum pH for the coagulation of most waters is at or slightly below 7.0. The optimum for various coagulants will usually be found to lie within the pH ranges shown in Table 83. The control of pH by the addition of sulphuric acid has been successful in the few plants that have used it; the addition of alkali has been equally successful and more extensive. In turbid waters that show rapid changes in pH it is difficult to determine the pH with sufficient rapidity and accuracy by the colorimetric methods. Under such conditions a recording potentiometer may be used with improved results.

In addition to the control of coagulation through observations of the various factors mentioned, it is desirable that periodic observations be made in the laboratory. These should determine (1) the first appearance of floc, (2) the appearance of the floc after 5 min. and after 30 min., and (3) the appearance of the supernatant liquor and the settleability of the floc at the end of the time corresponding to the retention period in the settling basin. A floc slightly larger than a pinhead is the most desirable size. Larger particles settle slowly, owing to the large surface in contact with the water, and they are fragile, tending to break into particles so small as to possess some colloidal characteristics.

In some plants it has been found advantageous to apply the coagulant at two or more points in the treatment of the water. This is known as double, or fractionated, coagulation. It may have the advantage of producing good results with less chemical than would be required by single coagulation, but more equipment and greater attention are required. It has not been extensively practiced.

TABLE 83.—SOME CHEMICALS USED IN WATER TREATMENT

Substance	Troublesome characteristics	Storage space, cu. ft. per ton	Storage container materials	Solution strength, per cent, and properties	Materials suitable for handling	pH range for best coagulation
Aluminum sulphate, $Al_2(SO_4)_3 \cdot 18H_2O$	Dusty	33: in bags	Concrete, steel, or wood	0.25–5.0[1] Acid and corrosive	Lead, rubber, acid-resistant bronze, cast iron	5.5–8.0
Ferrous sulphate, $FeSO_4 \cdot 7H_2O$	Cakes above 20°C. in moist air. Stains	31: in bags or bulk	Concrete, steel, or wood	0.25–6.0[1] Acid and corrosive	Rubber, lead, Duriron, and 18-8 stainless steel	8.5–11.0
Calcium oxide,[2] CaO....	Dusty. Slakes on standing in air	32: in bags or bulk	Concrete, steel, or wood	Up to 25[1] Alkaline and incrustant	Rubber, iron, cement	
Calcium hydroxide, $Ca(OH)_2$	Dusty	48: in bags or bulk	Concrete, steel, or wood	Up to 20[1] Alkaline and incrustant	Rubber, iron, cement	
Sodium aluminate, $Na_2Al_2O_4$	Hygroscopic	20 sq. ft. per ton in drums	Store in shipping containers	0.25–6.0[1] Alkaline	Rubber, iron	
Sodium carbonate, Na_2CO_3	Dusty	33; in bags or bulk	Concrete, steel, or wood	Up to 6.0[1] Alkaline	Rubber, iron	
Ferric chloride, $FeCl_3$....	Acid, corrosive, staining liquid	61: in bulk. 20 sq. ft. per ton in carboys	Rubber-lined concrete, steel, or wood	3–4 Acid and corrosive	Acidproof materials	5.0–11.0
Ferric sulphate, $Fe_2(SO_4)_3$	Stains	28; in bags or bulk	Concrete, steel, or wood	1–6.5 Acid and corrosive	Rubber-lined, Duriron, and 18-8 stainless steel	5.0–11.0
Sodium silicate, $Na_2SiO_3 \cdot Na_2O + SiO_2$	Alkaline and sticky	50 sq. ft. per ton in drums	Store in shipping containers	Up to 6	Rubber, and iron	

[1] Can be fed dry, also.

[2] Calcium oxide, or quick lime, expands on prolonged storage and may burst its containers.

500. Coagulants. A few of the many substances used in the coagulation of water are listed in Table 83. Among the more commonly used coagulants are aluminum sulphate (alum), ferrous sulphate (copperas or iron), sodium aluminate, and ferrous sulphate. Many chemicals and substances other than those listed in Table 83 have been used. Among these may be included calcium chloride; barium hydroxide; sodium sulphate; black alum, which is a combination of alum with about 2 per cent of activated carbon; many other chemicals; and various clays. It has been found that the addition of silica[1] in the form of a colloidal hydrous silicon dioxide, possessing a strong negative charge, materially shortens the time required to produce coagulation with aluminum or ferrous sulphate.[2] A more effective form of silica, known as activated silica, is a result of reaction between sodium silicate and sulphuric acid. The silica is an aid to coagulation but it is not a coagulant. The amount of silica, SiO_2, added should be approximately 40 per cent of the aluminum sulphate used. The use of silica is not universally approved as small amounts of silica in water may damage boilers. The approximate chemical compositions of some commonly used coagulants are shown in Table 84.

TABLE 84.—APPROXIMATE COMPOSITION OF COAGULATING CHEMICALS
(Percentage by weight)

Chemical constituent	Coagulating chemical			
	Potash alum	Sulphate of alumina	Sulphate of iron	Ammonia alum
Matter insoluble in water..	.....	0.30	0.50	
Alumina, Al_2O_3.........	10.77	17 00	.....	11.0
Iron, Fe_2O_3 and FeO.. ..	..	0 25	57.50	
Potash, K_2O....	9.93			
Sulphur trioxide, SO_3.........	33 76	38.70	28.80	26.0
Ammonium sulphate, $(NH_4)_2SO_4$.	.....	.....	.....	14.0
Aluminum sulphate, $Al_2(SO_4)_3$.......	.....	.....	....	37 0
Water...................... ...	45 54	43.75	13.20	0 5

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[1] See also HAY, H. R., "Water Purification Methods Involving Sodium Silicates," *Jour. A.W.W.A.*, June, 1944, p. 626, with bibliography; and HAY, H. R., "Improving Water Plant Operation, with Activated Silica," *Water and Sewage Works*, December, 1946, p. 479.

[2] See also BAYLIS, J. R., "Silicates as Aids to Coagulation," *Jour. A.W.W.A.*, Vol. 29, p. 1355, 1937; and BAYLIS, J. R., "Coagulation," *Water Works & Sewerage*, June, 1937, pp. 231 and 426.

501. Coagulation with Alum. In order that aluminum sulphate may react to form a precipitate, it is necessary that the water into which the alum is placed shall contain some natural alkalinity, usually in the form of calcium carbonate. The hypothetical reactions that occur are as follows:

1. Alum and natural alkalinity:

$$Al_2(SO_4)_3 \cdot 18H_2O + 3CaCO_3,H_2CO_3 = Al_2(OH)_6 + 3CaSO_4 + 6CO_2 + 18H_2O$$

2. Alum and soda ash:

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Na_2CO_3 + 3H_2O = Al_2(OH)_6 + 3Na_2SO_4 + 3CO_2 + 18H_2O$$

3. Alum and lime:

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Ca(OH)_2 = Al_2(OH)_6 + 3CaSO_4 + 18H_2O$$

That which actually happens is much more complicated than the foregoing reactions would indicate. The amount of alum necessary to give perfect coagulation cannot be foretold by chemical analysis or other hypothesis. Theoretically 1 grain per gal. of commercial alum requires 7.7 p.p.m. of alkalinity as $CaCO_3$; practically the amount required may vary between 4 p.p.m. for highly colored waters up to 6 to 7 p.p.m. Less than the theoretical amount is used because part of the aluminum ions appear to combine directly with the impurities in the water.

502. Practical Considerations in the Use of Alum. The absence of sufficient natural alkalinity to complete reaction (1) is unusual. In general, 8 to 10 p.p.m. of alkalinity as calcium carbonate will react with 1 grain per gal. of alum, forming aluminum hydroxide, although it is very well known that none of these reactions goes to completion, for most of them are reversible, each depending for its final equilibrium upon all the others. Where there is less than 20 p.p.m. of natural alkalinity as $CaCO_3$ in the water, about 0.35 grain of lime or 0.5 grain of soda ash should be added per grain of alum. The reactions are shown in (2) and (3). Where sodium carbonate or lime is added to increase the alkalinity of the water, the chemical should be thoroughly mixed with the raw water for a short time before alum is added. If added too long before the coagulant, some softening action may occur, necessitating the application of more chemicals. Doses used in practice vary between 0.3 and 2.0 grains per gal., depending on many factors the understanding of which requires a broad knowledge of the reactions involved.

The most common method of coagulating water with alum is to depend on the natural alkalinity in accordance with reaction (1). The use of soda ash, although giving better chemical results, is too costly to justify its general adoption. Where lime and soda ash are added, the basic sulphates of alumina, $Al_2(SO_4)_3(OH)_2$, $Al_2(SO_4)_3(OH)_4$, and $Al_2(SO_4)_3(OH)_6$, will be formed with incomplete coagulation, because of their solubility. This will result in a waste of the coagulant and an increase of the hardness of the water. A low temperature combined with low natural alkalinity is most conducive to this difficulty. The addition of large amounts of alum will make the conditions worse. Increasing the concentration of calcium or magnesium carbonate or of sodium bicarbonate in water, up to 60 to 75 p.p.m., will decrease the time required for coagulation. Beyond this amount the time required increases gradually as the concentration is increased.[1]

Alum is used occasionally in other forms, *e.g.*, as ammonia alum or as "black" alum. Ammonia alum is a mixture of sulphate of alumina and ammonium sulphate Because of its slow dissolving powers, it is not commonly used in waterworks, but this property makes it desirable in swimming pools and for the stabilization of chlorine by means of its ammonia radical. Black alum is used in a few plants[2] to obviate the necessity for the separate application of alum and activated carbon.

503. Difficulties with Alum. In reactions (1), (2), and (3) the aluminum hydroxide forms the insoluble flocculent precipitate that is desirable for purifying the water. It is to be noted[1] that some calcium sulphate remains in solution when the natural alkalinity or the addition of lime is depended on to react with the alum. This increases the permanent hardness of the water and is, therefore, detrimental. The amount of permanent hardness, expressed as calcium carbonate, resulting from this form of treatment is about 11.0 p.p.m., or about 0.6 grain per gal. for each grain of alum, $Al_2(SO_4)_3$, added. This amount is not objectionable in most water supplies. The increase of free carbon dioxide may be objectionable because of its corrosive nature. Unsatisfactory coagulation may result from the use of alum if sodium or potassium is present in the water. The presence of either of these substances spoils the precipitated aluminum hydroxide floc, causing it to appear in extremely fine particles which are almost colloidal. They will not settle, and they will pass through sand filters. The difficulty may be remedied by increasing the dose of alum.

If it becomes necessary to add alkalinity to improve coagulation, there is a danger of fixation of color of the water if the added alkali is too strong.[3] The reaction is seldom fully completed, and high concentrations of colloidal matter sometimes further hinder the reaction, with a result that residual alum remains in the water that goes to the filters. It is difficult to adjust the coagulation through pH control, because the optimum range of pH values is small and the application of alum depresses the pH.

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[1] BAYLIS, J. R., "Coagulation," *Water Works & Sewerage*, June, 1937, p. 426.

[2] COOK, H. M., "Results of the Use of Activated Carbon at Batavia, N.Y.," *Water Works & Sewerage*, February, 1934, p. 47.

[3] "The Correct Use of Coagulants," *Water and Water Eng.*, Mar. 31, 1934, p. 177.

504. Residual Aluminum Compounds. Water coagulated with alum contains aluminum compounds in solution. The exact nature of these compounds is so complex that it is the practice to report them as aluminum or aluminum oxide, Al_2O_3, by determining the concentration of aluminum ion, Al^{+++}, and converting to equivalent alumina by multiplying by 3.77. Residual alum cannot be reported in this manner, as it is incorrect to assume that the residual aluminum compound is aluminum sulphate. Since aluminum hydroxide is soluble in water to the extent of 0.3 to 0.5 p.p.m., some residual alumina is to be expected, but the amount should not exceed about 0.3 p.p.m. Colorimetric tests, using either the dye alizarin red or logwood, are satisfactory for routine operation control.[1]

505. Commercial Alum. Commercial alum is crystalline in form $Al_2(SO_4)_3 \cdot 18H_2O$ and contains about 18 per cent Al_2O_3. Emergency alternate specifications for sulphate of alumina are to be found in *Jour. A.W.W.A.* July, 1942, p. 1073, and tentative standard specifications in *Jour. A.W.W.A.*, December 1945, p. 1385. For use in water purification, commercial alum is sold in all quantities from a single bag to carload lots, both in packages and in bulk. Paper bags or paper-lined bags or paper-lined barrels are commonly used as a protection against atmospheric moisture. A 400-lb. barrel contains about 376 lb. net of alum crystals and occupies a space about 24 in. square and 29 in. high. Bags containing 200 lb. net of alum weigh about 202 lb. and occupy a space about 15 in. square and 30 in. high.

506. Coagulation with Ferrous Sulphate. Ferrous sulphate is a frequently used coagulant in water purification. It has an advantage over alum in that it may be less expensive and the floc is heavier and sinks more rapidly. An outstanding disadvantage is the need for using lime with it. More complete chemical control is required, and there are greater dangers from afterprecipitation in the distribution system due to the reaction between surplus lime and bicarbonate alkalinity. Ferrous sulphate is unsuitable for the treatment of soft colored waters because they are best coagulated at a pH below 7.0.[2] The color appears to become set by the addition of alkali to colored waters; hence the use of ferrous sulphate is limited to those waters in which alkalinity will not interfere with color removal. It is best suited to use in turbid waters of high natural alkalinity. There is usually insufficient alkalinity in natural waters to react with ferrous sulphate, so that lime must usually be added to produce a floc and in order to avoid soluble compounds of iron remaining in the treated water.

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[1] See also Cox, C. R., "Determination of Soluble Residual Aluminum in Water Coagulated with Alum," *Water Works Eng.*, Feb. 17, 1937, p. 217.

[2] Bartow, E., and others, "Formation of Ferric Floc by Ferric Coagulants," *Proc. A.S.C.E.*, December, 1933, p. 1529.

507. Reactions with Ferrous Sulphate. The chemical reactions that occur when ferrous sulphate and lime are used in coagulation depend, in part, on the order in which the chemicals are added to the water.

When the iron is added first, the reactions are

$$FeSO_4 \cdot 7H_2O + CaCO_3 \cdot H_2CO_3 = FeCO_3 \cdot H_2CO_3 + CaSO_4 + 7H_2O \quad (1)$$

$$FeCO_3 \cdot H_2CO_3 + 2Ca(OH)_2 = Fe(OH)_2 + 2CaCO_3 + 2H_2O \quad (2)$$

$$4Fe(OH)_2 + 2H_2O + O_2 = 4Fe(OH)_3 \quad (3)$$

When the lime is added first, the reactions are

$$FeSO_4 + Ca(OH)_2 = Fe(OH)_2 + CaSO_4 \quad (4)$$

$$4Fe(OH)_2 + 2H_2O + O_2 = 4Fe(OH)_3 \quad (5)$$

The ferrous hydroxide, $Fe(OH)_2$, forms a desirable, heavy gelatinous precipitate. It is, however, oxidized in accordance with reaction (3) to ferric hydroxide, $Fe(OH)_3$, likewise a satisfactory gelatinous precipitate.

508. Amount of Iron and Lime Required. As with other coagulants, the amount of ferrous sulphate required is most commonly determined by experience with the water being treated. The amount of lime required is affected by the amount of ferrous sulphate added and the amount of carbon dioxide present, which must be neutralized. A slight excess of lime, 1 to 5 p.p.m., should be added, if sufficient alkalinity is present, to enhance the precipitation of ferric hydroxide through the formation of calcium carbonate crystals about the ferric hydroxide. Too great an excess of lime should be avoided in order to prevent afterprecipitation effects. The amount of ferrous sulphate to be added is dependent, mainly, on the turbidity, the natural alkalinity, and the free carbon dioxide in the raw water.

It is more difficult to use lime and iron than to use alum alone because of the necessity for the careful adjustment of the amounts of lime and iron to secure complete and effective reactions. Such a method of coagulation requires competent and constant supervision.

509. Sodium Aluminate. Sodium aluminate, $NaAlO_2$, is an alkaline compound, a typical analysis of the best commercial grade being Al_2O_3, 55.5 per cent; combined Na_2O_3, 34 per cent; Na_2CO_3, 4.5 per cent; $Na(OH)$ (excess), 6.3 per cent. The use of sodium aluminate is advantageous under certain conditions because there is no need for any additional alkali. There is an immediate precipitation of all the aluminum hydroxide because of the rapidity of the reaction and the presence of alkali. It does not increase the noncarbonate hardness and can be mixed with lime and soda ash solutions, thus making it unnecessary to provide additional equipment for its application. Among other advantages claimed for it may be included the elimination of corrosive qualities of the water, a wide range of pH, quick flocculation, and no need for the addition of alkali. Its cost has been an important factor in preventing its wider adoption in practice.[1]

510. Ferric Coagulation. Ferric chloride, $FeCl_3$, ferric sulphate, $Fe_2(SO_4)_3$, and a mixture of the two, known as chlorinated copperas, are among the more recently recognized coagulants, their greatest field of usefulness being found, probably, in sewage treatment.[2] Among the advantages claimed for the use of ferric coagulants in water purification may be included:[3] (1) Coagulation is effective over a wider range of pH than with alum. (2) The time required for floc formation, conditioning, and settling is, in many cases, considerably shorter than that required for alum. (3) Filter runs have been increased in several tests. (4) Manganese is successfully removed at pH values above 9.0. (5) Very little iron is carried through with the effluent. (6) Hydrogen sulphide is removed, and tastes and odors are reduced. (7) There is a decreased tendency toward the formation of mud balls compared with aluminum floc. (8) Under some conditions the ferric coagulants are more economical than aluminum hydroxide.[4]

511. Ferric Chloride.[5] When ferric chloride is added to water, the following reaction occurs:

$$FeCl_3 + 3H_2O = Fe(OH)_3 + 3H^+ + 3Cl^- \qquad (6)$$

The ferric hydroxide is precipitated, forming a desirable coagulant, which is heavier than aluminum hydroxide and requires a shorter retention period and less careful adjustment of pH. Acidities that are encountered with some soft, highly colored waters make their coagulation with alum impossible, but they may be successfully coagulated with ferric compounds. Ferric chloride has been used with success also in waters containing hydrogen sulphide because, in contrast to alum, ferric chloride is an oxidizing agent, whereas alum is a reducing agent. In treating sulphide waters, the ferric salt is reduced to the ferrous state, after which it behaves like ferrous sulphate in the lime and iron process.

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[1] "The Correct Use of Coagulants," *Water and Water Eng.*, Mar. 31, 1934, p. 177.

[2] See also Hyde, C. G., "Practical Aspects of Coagulation with Ferric Chloride," *Jour. A.W.W.A.*, May, 1935, p 631.

[3] Black, A. P., "Coagulation with Iron Compounds," *Jour. A.W.W.A.*, 1934, p. 1713.

[4] Craig and Bean, "Ferric Iron Coagulation of Soft Water," *Water Works & Sewerage*, September, 1932, p. 301.

[5] Hyde, C. G., "Practical Aspects of Coagulation with Ferric Chloride," *Jour. A.W.W.A.*, May, 1935, p. 631.

512. Chlorinated Copperas. Chlorinated copperas is a mixture of ferric chloride and ferric sulphate prepared by adding chlorine to a solution of ferrous sulphate in the ratio of 1 part of chlorine to 7.8 parts of copperas. It is claimed that, as a result, the effectiveness of the copperas as a coagulant is increased and the necessity for high alkalinity is eliminated. In practice, copperas solution may be dosed with chlorine discharged from a chlorinator. Among the advantages claimed for the use of chlorinated copperas as a coagulant[1] are (1) a desirable floc formation with tough particles of floc resistant to breaking up; (2) floc formation usually settles well with only a small residual going to the filters; (3) the coagulating effect has a wide range of optimum pH from 8 or 9 to 6; (4) a compact floc of hydrated ferric oxide, which does not dissolve in alkaline waters, is formed at all pH values above 3.5; and (5) the coagulant is particularly effective in color removal compared with the relative ineffectiveness of ferric and ferrous hydroxides on colloids having an isoelectric point below 7.0.[2] Chlorinated copperas has not been widely used in practice so that experience with it is restricted.

513. Handling Ferric Coagulants. Solutions of ferric chloride are extremely corrosive and must, therefore, be stored and transported in nonmetallic materials or certain of the newer alloys of iron which have been reported to be resistant to its corrosive powers.[3] The substance has a highly astringent action on wood, making it necessary frequently to tighten wooden storage tanks. In computing the allowable storage space for the crystals, which are highly deliquescent, an allowance of 55 to 60 lb. per cu. ft., or 70 sq. ft. per ton, should be allowed. Because of its hygroscopic qualities, ferric chloride cannot be applied through a dry-feed machine, necessitating the preparation of an aqueous solution. In general, the strength of the solution may vary from about 2 per cent in very small plants to 20 per cent, or even greater, in large, well-equipped plants. The strength used depends on the rate of use and the precision with which dosing is to be accomplished.

Ferric sulphate is supplied commercially in a concentrated form, ranging from 62 per cent $Fe(SO_4)_3$ to anhydrous. It is not deliquescent, and it is less corrosive than ferric chloride and can, therefore, be stored and handled with greater ease. On going into solution, considerable heat is generated, which assists solution when the volume of water is not too great. Storage solutions should not be made in strengths of less than about 1 per cent to prevent rapid hydrolysis of the coagulant. Hydrolysis is practically instantaneous and will occur at pH of 5.0 to 6.0.

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[1] Billions, L. C., "Experience with Chlorinated Copperas as a Coagulant," *Water Works & Sewerage*, 1934, p. 73.

[2] Hedgepeth, L. L., "Chlorinated Copperas Used in Water Purification," *Ind. Eng. Chem.*, September, 1931, p. 534.

[3] Potter, J. M., "Practical Aspects of Storing and Handling Ferric Chloride," *Water Works & Sewerage*, January, 1935, p 12.

514. Coagulation with Clay. Coagulation with clay has been experimented with in a few water purification plants, but it is rarely used in practice. The function of the clay is probably to provide colloidal nuclei for the adsorption of finely divided and colloidal matter in the water. Only special forms of clay are suitable for the purpose, and the cost of the process depends on the availability of suitable materials. Leipold[1] reports that the use of clay materially increases the length of filter runs that had been shortened by reason of microscopic organisms in the raw water and aids in checking the shrinking of sand in the filters. He applied the clay through a dry-feed machine at rates varying between 1.1 and 7.1 grains per gal., with a mixing period of 15 min. and a coagulation period above 2 hr. At Chester, Pa., Dodd[2] used an average dose of only 0.3 grain per gal. with a range between 0.1 and 0.6. The cost of the clay is in the neighborhood of $\frac{3}{4}$ cent per pound.

Bentonite, which swells greatly on wetting, has the property of exchanging the alkali and the alkaline-earth ions,[3] and it is possible that the differences in colloidal properties of various clays may be due to variations in composition resulting from such exchange of bases. It has been found by test that the coagulating effects of bentonite are not materially affected by the pH value of the water and that with equivalent dosages bentonite flocs are uniformly more voluminous and more rapid in settling the alum flocs.

515. Lime Treatment. Although not a form of coagulation, an excessive amount of lime, up to 2 to 3 grains per gal., is sometimes added to water before filtration in an attempt to reduce the bacterial load on the filters, to avoid chlorination after filtration, and to aid in water softening. Lime treatment of the effluent from a rapid sand filter plant is sometimes employed to overcome the corrosive qualities resulting from the presence of carbon dioxide and oxygen. The lime treatment should be sufficient to neutralize the carbon dioxide and to permit the formation of a protective coating on exposed metallic surfaces.

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[1] LEIPOLD, CARL, "Clay Used as Artificial Turbidity to Eliminate Short Filter Runs," *Water Works & Sewerage*, August, 1933, p 304.

[2] DODD, R. L., "The Use of Absorbing Clay for Removing Taste and Odor Producing Substances from Water," *Jour. Penna. Water Works Operators' Assoc.*, 1932, p. 65.

[3] OLIN, H. L., and H. W. PETERSON, "The Use of Bentonite as a Coagulant," *Jour. A.W.W.A.*, April, 1937, p. 513.

516. Costs and Weights of Chemicals. In general, the relative costs of the different coagulants are as listed below, the least costly being stated first: (1) lime and iron, (2) alum alone, (3) alum and lime, (4) alum and soda ash, (5) sodium aluminate. The approximate relative costs, per 100 lb., of the various chemicals are shown in Table 85.

TABLE 85.—RELATIVE COSTS OF CHEMICALS

Chemical	Relative cost per pound	Pounds per cubic foot
Alum	1 10	60 0
Ferrous sulphate	0 70	66 0
Lime	0 35	36 0(CaOH)
Soda ash	1 00	42 0
Sodium aluminate	2 70	
Ferric chloride	2 05	
Bleaching clay	0 75	
Activated carbon		13 5
Activated alum		58 0

517. Handling and Storing Chemicals. Dry chemicals are delivered to water-purification plants in paper or burlap bags, in wooden barrels which are usually paper lined, and in bulk, sometimes in paper-lined box cars. The chemicals should be stored in a dry place at a moderate and fairly uniform temperature. These conditions are necessary to prevent deliquescence and caking. Temperatures above 70°F. are undesirable. It is frequently desirable to store chemicals on the floor or floors above the chemical preparation room so that gravity may be used in the preparation of the solutions or charging of the feeding devices. Chemicals in bags are usually piled on the floor, two bags high. Those in bulk should be stored in bins so designed and located that they can be withdrawn from the bottom of the bins and delivered where desired. The bins should be relatively deep, exposing but a small amount of chemical to the atmosphere. The bottom slope should not be less than 45 deg. from the vertical. Corrosive, volatile, or deliquescent chemicals, including such chemicals as ferric chloride, both liquid and solid, sodium aluminate, sodium hydroxide, ammonium chloride, and liquid and gaseous materials, must ordinarily be stored in shipping containers.

Equipment provided for the handling of chemicals should include hand trucks, elevators, overhead tracks, cranes, mechanical conveyers, and pneumatic conveyers. The last named are particularly useful in handling dry, dusty chemicals such as hydrated lime.

518. Preparation of Solutions. In the preparation of chemical solutions a satisfactory method, with chemicals that are easily dissolved, such as alum, ferrous sulphate, or soda ash, is to place the chemical in a metal basket, perforated-concrete box, or perforated wooden box and to spray warm water over the chemical. The water, containing the chemical in solution, is retained in solution-storage tanks. The storage tanks may be large enough to hold sufficient solution for one operation shift, or large enough to dissolve the contents of one shipping container. The first size may be conducive to operation responsibility and efficiency, the latter to convenience and safety in operation. Another satisfactory method is to use the solution pot, a container into which the chemicals are placed and through which the water flows, as indicated in Fig. 225. Enough chemical is placed in the pot to maintain a saturated solution in the effluent pipe. The rate of flow of solution is proportional to the rate of flow through the main water pipe.

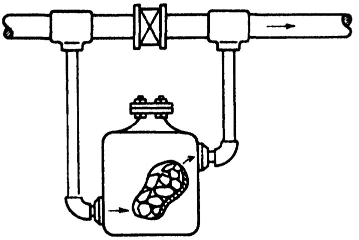
FIG. 225. Solution pot for chemical feed

Recommended strengths of solutions are given in Table 83. Solutions for application to the water should be more dilute than storage or "stock" solutions, in order to increase the delicacy of control of dose. However, too high a dilution is undesirable because of the possibility of premature chemical reactions and because of the bulk of solution to be handled.

519. Storing Chemical Solutions. Containers for the storage of chemical solutions are made of wood, steel, concrete, glass, rubber, brass, copper, lead, and metallic alloys. Materials to be used for storing various chemicals are listed in Table 83. Unprotected wood and steel are not widely used because of their short life in the presence of most chemical solutions used in water purification. They may be protected by acid-resistant paint, bituminous compounds, or lead lining, or the steel may be enameled.

Weak acid solutions may be stored in concrete tanks, enameled iron tanks, or lead-lined tanks. Alkaline solutions may be stored in iron tanks. Lime should be slaked only in an iron tank as the heat generated will injure most other materials. Hypochlorite of lime should be stored in rubber-lined or vitreous tanks.

520. Stirring. The stirring of most solutions is essential to prevent settling and stratification. It can be accomplished by slowly moving paddles in the tank. There should be at least two paddles in the tank, revolving in opposite directions, to avoid the creation of a vortex. Air agitation may be used for chemicals that will not be affected thereby. Ferrous sulphate must be protected against oxidation even when paddles are used. It is not essential that the stirring mechanism should be operated continuously for all solutions.

521. Chemical Feed. Chemicals may be fed either in solution or in the form of the dry chemical. Dry feed is generally preferred because of the freedom from the additional equipment required for the preparation, storage, and feeding of solutions. However, it may be desirable to use solution feed in small plants where the cost of dry-feed machines may not be economically justified, or where the chemical to be fed is deliquescent or is otherwise unsuitable for handling in a dry-feed machine.

522. Solution-feed Devices. Solution-feed devices may be either continuous rate or proportional feed. A common type of continuous-rate feed device consists of an orifice in the bottom of a small solution-feed tank. Either or both the size of the orifice and the head on it are adjustable, by the operator, to give any rate of flow desired within the limits of capacity of the control device.

Feed of chemical solution as a constant proportion of the rate of flow of water through the plant can be controlled by (1) a reciprocating solution-feed pump synchronized with the main pumps, (2) by devices depending on the principle of the venturi tube, (3) by devices operating on the principle of the bucket conveyer, (4) by continuous pH analyzers that control solution flow electrically, and (5) by other ingenious devices. Desirable features of solution-feed devices include simplicity in design, resistance to corrosion, and freedom from clogging.

523. Pipes for Chemical Solutions. Solution-feed devices should be arranged, where possible, so that the solution falls into the raw water by gravity, thus avoiding the need for pipes or pumping. Materials used for pipes are the same as those used for storage tanks and are listed in Table 83. Lead, bronze, and brass are suitable for conveying alkalies, although brass is not commonly used. Rubber, in the form of chemical hose, seems to stand up well under any solution, and where flexible hose is used stoppages may sometimes be cleared by bending and pressing the hose.

Pipe lines should be laid on a continuous slope, as steep as possible, leading without bends, to cleanouts. Air traps in the lines must be avoided. Where bends are necessary cleanouts should be provided so that a rod can be inserted in the pipe to or beyond the next cleanout. Straight runs may have cleanouts at about 20-ft. intervals. Provision should be made for drainage at cleanouts. A wye, tee, or cross makes a satisfactory cleanout. Provision should also be made for flushing the pipe with water under pressure; duplicate pipes are sometimes desirable to permit cleaning of one while the other is in service.

524. Dry-feed Apparatus. Dry-feed devices are desirable because of simplicity, relatively small space required, neatness, and freedom from corrosion. All chemicals cannot be handled in dry-feed apparatus because of caking, clogging, and deliquescence. The desirable characteristics of a chemical that is to be fed dry are uniformity of size of grain, constancy in composition, permanency under prevailing conditions of temperature and pressure, and freedom from being hygroscopic or efflorescent. Aluminum sulphate is fairly fine and uniform and gives little trouble. The dry feeding of alum is especially advantageous, as it avoids the corrosiveness of alum solutions on the pipes that convey it. Hydrated lime is more troublesome than alum because of its tendency to bridge orifices and to take up moisture. Iron sulphate is sometimes quite troublesome, as the number of molecules of water of crystallization depends on the temperature, with the result that, in changing temperature, stored iron sulphate may become a solid mass in the bin.

Of the two types of chemical-feed regulators, volumetric control is more common than gravimetric control. A common form of chemical-feed machine is shown in Fig. 226. Arching of the chemical in the hopper is prevented by vibratory or internal mechanical agitation.

Dry-feed machines are preferably driven by constant-speed electric motors. Although the amount of power required is very small, water motors are undesirable because of fluctuations in water pressure. Rate of feeding can be adjusted by controlling the space between the hopper and the tray and by controlling the intensity of vibrations of the tray. Without auxiliary gravimetric control, volumetric feeders need close attention to assure accurate feeding. Dry-feed machines are available that are dependable and accurate in their performance. They may be equipped with rate-of-feed gages and be suspended on recording scales so that a check of their performance can be maintained.

525. Factors Affecting Coagulation. Among the principal factors affecting coagulation are the following:

1 Quantity and kind of coagulant
2. Characteristics of the water, including suspended matter, temperature, and pH.
3 Time, violence, and method of mixing

Temperature has more influence on the rate of formation of floc than it has on the amount of coagulant required. However, where rate of flocculation is too slow for proper plant operation, it may be necessary to increase the chemical dose. Optimum pH ranges for some coagulants are shown in Table 83.

526. Mixing, Conditioning, or Flocculation. After the chemical solution or the dry chemical has been added to the raw water, the two should be thoroughly and quickly mixed and the mixture should be prepared, or conditioned, for the following period of quiescent sedimentation in the settling basin, commonly known as the coagulating basin. The term mixing is used to designate the action of bringing the chemical into contact with all portions of the water, and the term conditioning, or flocculation, refers to the process of accretion or agglomeration of the particles of floc into masses of sufficient weight and bulk so that they may settle readily in the coagulating or sedimentation basin. In the operation of a plant it is not possible to distinguish sharply between mixing and conditioning, as they may proceed simultaneously. Thorough mixing of the coagulating chemical with the water is accomplished by a short period of violent agitation. This should be followed by a longer period of conditioning, involving gentle stirring, during which the desired floc is developed.

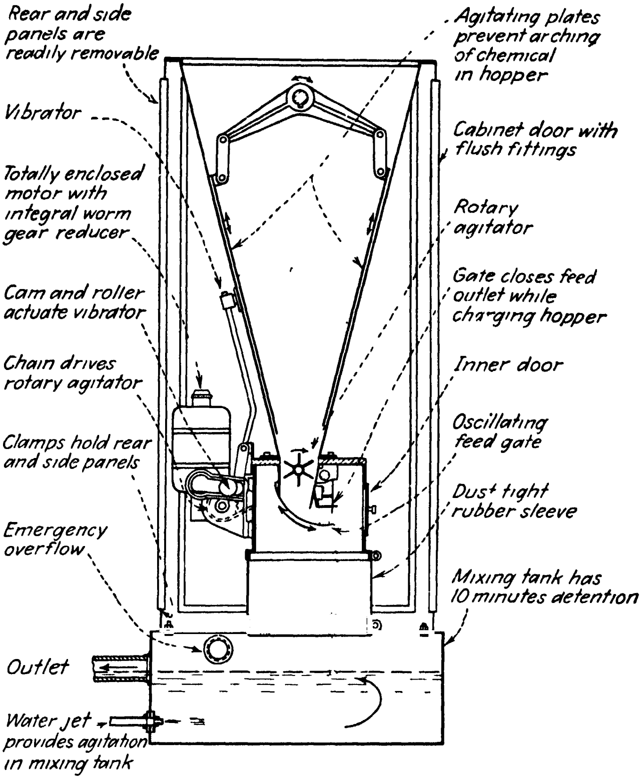
Rear and side panels are readily removable
Vibrator
Totally enclosed motor with integral worm gear reducer
Cam and roller actuate vibrator
Chain drives rotary agitator
Clamps hold rear and side panels
Emergency overflow
Outlet
Water jet provides agitation in mixing tank
Agitating plates prevent arching of chemical in hopper
Cabinet door with flush fittings
Rotary agitator
Gate closes feed outlet while charging hopper
Inner door
Oscillating feed gate
Dust tight rubber sleeve
Mixing tank has 10 minutes detention

FIG. 226 – Diagrammatic section through Permutit oscillating dry chemical feeder. For alum and similar free-flowing chemicals the rotary agitating rakes and vibrator are omitted and the mixing tank is lined for protection against corrosion. (*Courtesy of the Permutit Co.*)

Factors to be considered in the design and control of mixing and flocculating devices include the time of retention and the velocity of the currents. In general, the time of mixing should be as short as possible, consistent with satisfactory results. The aim is to secure, as nearly and as quickly as possible, a thorough diffusion of the chemical throughout the incoming water. The duration of the period of flocculation depends on the condition of the raw water, the kind and amount of chemical used, the type of flocculator used, and the desired results. It can be determined best by laboratory control and experience with the water in hand. In general, periods of flocculation used in practice are in the neighborhood of 30 to 60 min. when preparing the water for coagulation basins and slightly longer for lime softening.[1] It is to be appreciated that, in general, a good flocculation is to be expected in a shorter time with highly turbid waters than is required for an equal amount of coagulant when the turbidity is low. Velocities of flow in mixing basins as high as 3 f.p.s. are sometimes used, and in flocculation about 0.5 to 1.0 f.p.s. Care should be taken to keep the floc in suspension with just enough agitation to prevent breaking it up. The type of motion seems of greater importance than the velocity of flow. A circulatory, twisting, sweeping confusion of the floc, resembling snow swirls, has been found most advantageous.[2] Care must be taken to form neither a large and feathery but fragile floc nor a tough and tenacious floc that lacks adsorbtive capacity.

Index Number	Corresponding Description of Floc
0	Colloidal floc. Absolutely no signs of agglomeration
2	Faint. Floc in minute particles not discernable by the ordinary observer
4	Distinct. Floc well formed but uniformly distributed
6	Decided. Floc large in size and about to precipitate
8	Excellent. Deposition of floc well established but not complete
10	Complete. Floc practically all deposited. Supernatant liquid practically clear

527. Measurement of Floc. Three recognized methods have been proposed for the rapid routine estimation of floc: the dissolved-color method of Nasmith,[3] Baylis's floc detector,[4] and the cover-glass method of Willcomb.[1] To aid in the control of flocculation Willcomb has proposed a floc index number as shown on page 454.

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[1] "Spring-Summer Operation at Lake Michigan Filter Plants," *Eng. News-Record*, Aug. 31, 1933, p. 250.

[2] SMITH, M. C., "Improved Mechanical Treatment of Water for Filtration," *Water Works & Sewerage*, April, 1932, p. 103

[3] NASMITH, G. G., "A Simple Color Test of Floc in Treating Colored Water," *Jour. A.W.W.A.*, Vol. 22, p. 396, 1930

[4] BAYLIS, J. R., "Sensitive Detection of Suspended Matter and A Proposed Standard of Clarity in Filtered Water," *Jour. A.W.W.A*, Vol. 11, p. 824, 1924.

In order to resist passage through pores of a filter, floc must be both large and strong. Factors affecting strength may be formulated into a floc index as he^3/d, in which h is the head necessary to force the floc through a bed of sand of uniform size e and depth d. Where the uniformity coefficient is low, e may be the effective size. Where e is expressed in millimeters and other dimensions are in feet, floc indices of 1 or less are weak and 5 is attainable and desirable.

528. Mixing Devices. Mixing devices include (1) pumps, (2) baffled basins, (3) spiral-flow basins, (4) basins with mechanically driven paddles, (5) air agitation, and (6) the hydraulic jump. Mechanical stirring devices provide the most satisfactory form of mixing device. Circular tanks with hopper bottom, equipped with revolving paddles and providing a mixing period of at least 30 min., are frequently used. The speed of the paddles should be adjustable to permit flexibility and the maintenance of the best speed in operation. The hopper bottom is provided to permit the removal of sludge which should not be allowed to accumulate. Where low-lift pumps are available, they are sometimes used as mixing devices.

Factors to be considered in the design of mixing devices include time, temperature, velocity of flow, quantity and character of chemicals added, violence of agitation, and characteristics of the water to be treated. These factors are generally interdependent; *e.g.*, water with a turbidity of 10, treated with 6 p.p.m. of aluminum sulphate (anhydrous), may require 60 min. mixing at a low temperature, but if the temperature, the concentration of chemical dose, and the violence of agitation are increased individually or collectively the period of mixing may be reduced to 10 min. Low temperatures delay floc formation. Agitation expedites it. Velocities of 1 to 3 f.p.s. through baffled basins are recommended. The value of prolonged mixing has been firmly established through experience in operation.

529. Baffled Mixing Basins. Basins through which water flows horizontally, back and forth, past around-the-end baffles placed 2 to 3 ft. apart, or in which it flows up and down, past under-and-over baffles about 2 to 3 ft. apart, are known as baffled mixing basins. They may serve as combined mixing and coagulating basins, depending on the turbulence of flow. Their use is becoming less general because of the advantages of mechanical mixing and mechanical flocculating basins.

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[1] WILLCOMB, G. E., "Floc Formation and Mixing Basin Practice," *Jour. A.W.W.A.*, Vol. 24, p. 1416, 1932.

Disadvantages of baffled basins, when compared with mechanical devices, include (1) less flexibility of control; (2) greater loss of head, which may amount to 1 to 2 ft. in the baffled basin and to practically nothing in the mechanical flocculator; and (3) greater expense of construction due to the baffle walls. Loss of head in around-the-end baffled basins, for 180 deg. turn of direction of flow, can be expressed as $3.22V^2/2g$, where V is the mean velocity of flow.[1]

530. Spiral-flow or Tangential-flow Basins. In circular basins the mixing or flocculating effect may be obtained by admitting the water tangentially at the periphery and withdrawing it at the center. The average induced rotating velocity in such tanks, with diameter not greater than about 40 ft., lies between 0.5 and 0.75 f.p.s., with an energy requirement equivalent to about 1 ft. of head loss. The depth is usually equal to or greater than the diameter. Disadvantages of such tanks include (1) short circuiting, (2) flow variations affect the intensity of agitation velocity, (3) high velocities at the periphery and low velocities at the center, and (4) velocities are not easily adjusted to meet plant conditions.

531. Mechanical Flocculators. Mechanical flocculators are displacing other forms of flocculators because they provide numerous gentle contacts between the flocculating particles that are essential to successful floc formation. Such contacts occur in a manner not provided by other methods of flocculation. Among the advantages of mechanical flocculators may be included (1) reduction of 10 to 40 per cent of the amount of chemical required by a baffled basin, (2) better floc formation with resulting diminution in required coagulating basin capacity, (3) less filter washing, (4) cost of installation lower than baffled basins, (5) flexibility of operation through control of speed of paddles, (6) good control of currents, (7) relatively low head loss or power requirements, (8) ease of installation in existing plants, and (9) marked improvement in treatment with activated carbon. Among the disadvantages may be included (1) low velocity near paddle shaft, (2) dead spaces in corners, (3) need of equipment maintenance, and (4) bad short circuiting.

Mechanical flocculators may be either circular tanks with paddles revolving on a vertical shaft, or rectangular tanks with paddles revolving on a horizontal shaft. Circular basins are used with paddles revolving about a vertical shaft with the direction of revolution of adjacent paddles opposed, or with some fixed and some moving paddles, to prevent the production of a vortex in the basin. Such flocculators are more commonly used in small plants and in the laboratory.

A common type of mechanical flocculator consists of a horizontal continuous-flow rectangular basin into which the water enters at the bottom, either at the side or middle of one end, and from which it leaves at the opposite side or middle of the other end, inducing a spiral flow. In passing through the basin the water encounters one or more slowly revolving paddles. Paddle areas, depending on the type of paddle, may vary between 10 and 25 per cent of the vertical cross-sectional area of the tank. The velocity of the paddle should be as fast as practicable without breaking up the floc.

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[1] See also "Manual of Engineering Practice," No. 19, "Water Treatment Plant Design," American Society Civil Engineering, 1941.

The peripheral speed of the paddle may be varied between 0.5 and 2.0 f.p.s. In some designs the paddles are driven at different speeds, those nearer the inlet of the basin revolving more rapidly, with the size of the blade being reduced as distance from the inlet increases.[1] The power requirements for such paddles has been found to lie between 3 and 6 kw.-hr. per million gallons of water treated for a 60-min. period of flocculation.

Velocities of flow used in practice, lie between 0.5 and 0.7 f p.s., with satisfactory periods of retention varying between 10 and 60 min.[2] Speeds and periods of retention provided in design should be adjustable within a fairly wide range by the operator to permit operation under optimum conditions. One form of horizontal-shaft flocculator paddle is shown in Fig. 227.

In the design of flocculator paddles for a horizontal-flow rectangular basin the length of the horizontal shaft is placed at right angles to the direction of the flow of water, and the paddles are spaced so that the shortest distance between tips is not greater than 2 to 3 ft. The paddles revolve in a direction such that the top moves toward the outlet of the basin with their tips just below or protruding an inch or so above the water surface.

532. Agitation with Air. Floc conditioning may be accomplished by blowing air from a grid of perforated pipes or other type of diffuser placed on the bottom of the basin through which the water is flowing.[3] Depths of 7 to 12 ft. are satisfactory, with a maximum practical depth of about 15 ft. Porous tubes, porous plates, or perforated pipes may be used for air distribution. Where perforated pipes are used laterals may be spaced 3 to 5 ft. apart, with $^1\!/_{16}$-in. diameter perforations on 3- to 6-in. centers. The distribution pipes, or porous distributors, should be placed in valleys with concrete ridges between them. About 0.5 cu. ft. of air per minute per square foot of tank area may be required.

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[1] See also SMITH, M. C., "Improved Mechanical Treatment for Water Purification," *Water Works & Sewerage*, April, 1932, p. 103.

[2] See also "Manual of Engineering Practice," No. 19, "Water Treatment Plant Design," American Society Civil Engineering, 1941, p. 29.

[3] See also WILLCOMB, G. E., "Floc Formation and Mixing Basin Practice," *Jour. A.W.W.A.*, 1932, p. 1416.

Advantages of air agitation include (1) flexibility of control, (2) low first cost and ease of installation, and (3) aeration combined with mixing. Among the disadvantages are (1) uneveness of agitation between bottom and top of basin and (2) clogging of air pipes and diffusers.

533. Coagulation Basins. The coagulation basin is designed to receive the water from the mixing or flocculating device and to retain it a sufficient length of time to permit the sedimentation of the proper amount of floc. If the water is to flow to a rapid sand filter, sufficient floc should remain in the effluent from the coagulating basin to form a light mat over the surface of the filter. The velocity of flow in the coagulation basin should be as slow as possible, not to exceed 3 f.p.m., and preferably less than 1 f.p.m. The basin should be designed to avoid turbulence and to provide a uniform velocity throughout. The principles of design in these respects are the same as for a plain sedimentation basin.

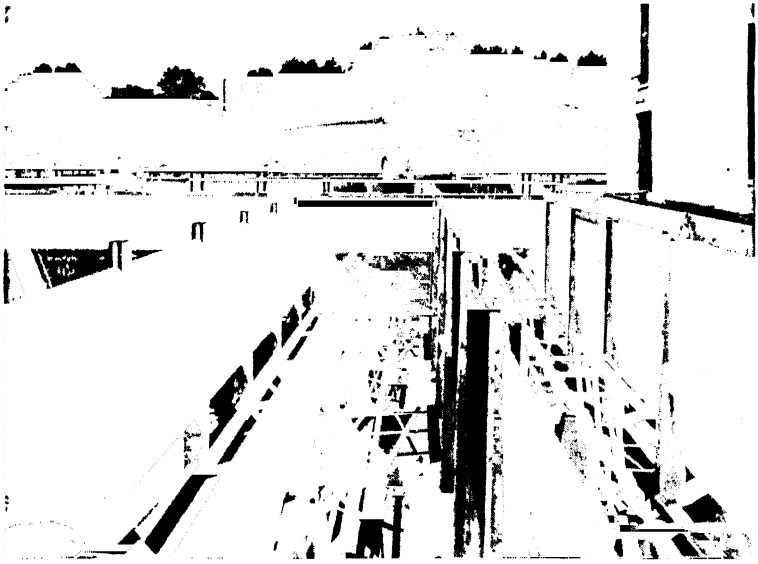

FIG. 227 – Flocculator paddles, Lexington, Ky. (*Courtesy of J. Stephen Watkins.*)

Periods of detention used in practice vary between wide limits, but the most common periods lie between 2 and 6 hr., the shorter one being more frequently used.

Coagulation basins may be equipped with continuous scrapers for the removal of deposited floc, operating similarly to those described in Sec. 497. They are finding considerable favor in waterworks practice because of their efficiency, the improved results obtained by the continuous removal of sludge, the possibilities of sludge recirculation, and the prevention of putrefaction of the sludge.

534. Upward-flow Coagulation. In upward-flow coagulation basins, suspended particles are removed by passing the flocculated water upward through a blanket of previously precipitated particles. Clear water is taken out at the surface and the particles, caught in the suspended blanket, settle to the bottom and are removed as sludge. Upward velocities of about 0.25 to 0.3 f.p.m. have been used. Several types of proprietary devices are in use. They are used not only in coagulation, but in softening and in the clarification of colored and turbid waters. One type, a Spaulding precipitator, is illustrated in Fig. 228.

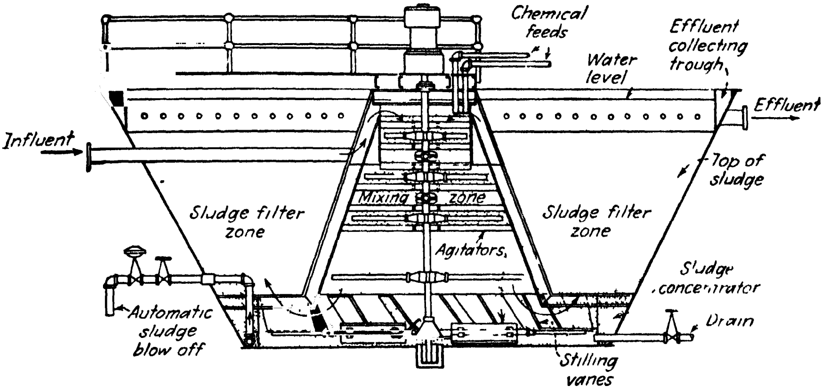

FIG. 228.—Section of Permutit (Spaulding) precipitator

CHAPTER XXVIII

SAND FILTRATION

535. Theory of Filtration. The common method for filtration of water is to pass it through a layer of sand. It has been found by experience that, by passing water through sand, suspended and colloidal matter are partially removed, the chemical characteristics of the water are changed, and the number of bacteria is materially reduced. These phenomena are explained on the basis of four actions: mechanical straining, sedimentation or adsorption, biological metabolism, and electrolytic changes.

Mechanical straining removes the particles of suspended matter that are too large to pass through the interstices between the sand grains. It cannot remove colloidal matter or bacteria too small to be strained out.

Sedimentation and adsorption account for the removal of colloids, small particles of suspended matter, and bacteria. The interstices between the sand grains act as minute sedimentation basins in which the suspended particles settle upon the sides of the sand grains. These particles adhere to the grains because of the physical attraction between the two particles of matter and because of the presence of a gelatinous coating formed on the sand grains by previously deposited bacteria and colloidal matter.

Biological metabolism is the growth and life processes of living cells. This, together with electrolytic actions, causes chemical changes that occur in a water filter. Matter is used in the metabolism of all organisms. The chemical combinations of this matter are altered by the growth of the organism. Since there are living organisms on sand filters, it follows that the chemical make-up of the matter put upon the filter will be altered by the growth of the organisms on the bed.

A very brief summary of the ionic theory, to explain the electrolytic changes, would state that, when two ions with opposite electrical charges are brought into contact, the electrical charges are neutralized or so altered as to form different chemical substances. The electrical charges on the sand filter, and on the ionized matter in the water, react to alter the chemical constituents of the water. Ultimately, the electrical charge on the filter is exhausted, and the filter must be cleaned to renew these charges.

The action of a filter occurs principally at the surface of the sand in the layer of matter deposited thereon. Other actions take place in the gelatinous coating on the sand grains within the body of the filter. The surface layer contains a zoogloeal jelly in which the biological activities are at their highest. This layer of material is called the *schmutzdecke* (dirty skin). The successful operation of slow sand filters in particular is dependent on it. Until the *schmutzdecke* is built up, the bacterial removal may be low. This building up constitutes the growth of the gelatinous coating on the sand grains within the filter. It is referred to as the "ripening" of the filter. It may require some time after a filter has been put into service before the desired efficiency of bacterial removal is secured. Frequently a satisfactory effluent can be obtained immediately. The character of the raw water, the rate of filtration, and other factors determine this. As the filter ages, the pressure necessary to force the water through the *schmutzdecke* and the gelatinous coating within the body of the filter becomes so great as to necessitate the cleaning of the filter.

Relatively little or no *schmutzdecke* is formed at the surface of a rapid sand filter. The filter may be penetrated deeply by floc. The depth of penetration is dependent on such conditions as the rate of filtration, the head loss through the filter, the density of the suspended matter in the water, the porosity of the filtering material, the temperature, and the floc indices described in Secs. 526 and 527. In general, under satisfactory conditions of operation, floc should penetrate not more than 2 to 6 in., after which the head loss through the filter should increase rapidly, necessitating the washing of the filter.

536. Types of Sand Filter. Two general types of sand filters are used for public water supplies, the slow sand filter and the rapid sand filter. A slow sand filter is a device for the purification of water consisting of a layer of sand through which water is filtered at a relatively slow rate, the filter being cleaned by scraping a thin layer of dirty sand from the surface. A rapid sand filter is a device for the purification of water consisting of a layer of sand through which water is filtered at a more rapid rate than is practicable for use with a slow sand filter, the filter being cleaned by reversing the flow of water through the sand, sometimes supplemented by mechanical or air agitation during washing.

537. Comparison of Slow Sand and Rapid Sand Filtration. The outstanding features of operation that distinguish the slow sand from the rapid sand filter are the rate of filtration, the method of cleaning, and the absence of preliminary chemical treatment of the water in slow sand filtration. In comparing the two types of filters it is found that the slow sand filter is less likely to go wrong under inexperienced operation; it does not require such skilled attendance; the amount of head consumed is less; it is preferred by some because of the greater reliability of the removal of bacteria; the operating costs may be less per unit volume of water treated; and it is best adapted to waters low in color, turbidity, and bacterial count. Among the features of rapid sand filters that make them more desirable than slow sand filters may be included lower first cost; smaller area of land required; the effluent is clearer, has less color, and is sparkling; a smaller amount of sand is required for construction; the method of cleaning leaves the filter out of service for a few minutes only; treatment can be more quickly adjusted to variations in raw water quality; and, finally, the capitalized cost of the plant is less.

Slow sand filters were the first type developed for water purification, dating back to 1829. The first rapid sand filter was constructed at Somerville, N.J., in 1884. Since then the construction of rapid sand filters has almost supplanted slow sand filter construction in the United States, and the ratio of rapid sand to slow sand filters in other countries has steadily increased.

Both types of filter constitute only a part of the water purification plant of which they are the most essential feature.

538. Cost of Filtration. The first cost of a water filter, as of other engineering structures, depends on many factors of design, construction, location, etc. In general, the first cost of rapid sand filters constructed before the Second World War varied between $25,000 and $65,000 per million gallons daily capacity, the lower figures being applicable to the larger plants. Operating costs ran between $10 and $16, or even higher, per million gallons of water filtered. The first cost of slow sand filters varied between $45,000 and $100,000 per million gallons daily capacity with operating costs varying between about $5 and $12 per million gallons filtered. These costs are based on data presented by G. A. Johnson in the *Journal of the American Water Works Association*, Vol. 1, page 69, 1914, and are corrected to 1937 by an index of 2.0, together with supplemental published cost data.[1]

539. The Parts of a Slow Sand Filtration Plant. The parts of an ordinary slow sand filtration plant may include some or all of the following, named in the order in which the water to be treated passes through them:

1. Intake and coarse screen.
2. Devices for placing water at the necessary elevation. These may be pumps or gravity conduits.
3. Plain sedimentation basin.

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[1] See also "Water Quality and Treatment," Manual American Water Works Association, 1940, p. 199.

4 Filter.
5. Clear-water storage basin.

The course of water through a slow sand filter plant is shown in Fig. 229, and a plan and section through such a plant are shown in Fig. 230. The principal appurtenances of a slow sand filter plant which are not connected to any of the foregoing parts of the plant are the sand-storage

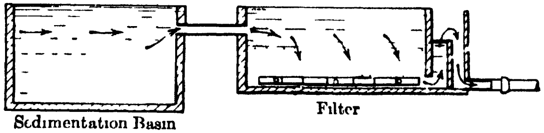

FIG 229 – Diagrammatic representation of path of water through a slow sand filter

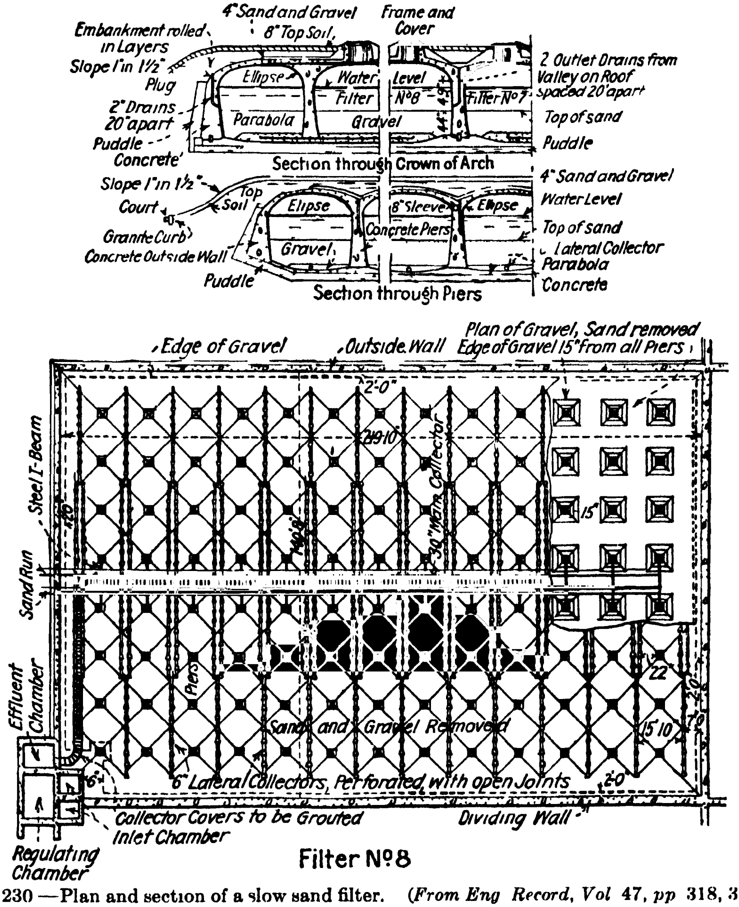

FIG. 230 —Plan and section of a slow sand filter. (*From Eng Record, Vol 47, pp 318, 366*)

and the sand-washing equipment. Water purification plants operating on the slow sand principle may include other steps in the treatment of the water, such as aeration, chlorination, and removal of algae, but these are not, strictly speaking, a part of a slow sand filter plant. It is to be noted that in an ordinary slow sand filter plant no provision is made for the addition of any chemical and that no coagulation is required.

The coarse screen placed at the intake to remove large floating and suspended matter is a protection to the pumps, conduits, and filters. The sedimentation basin is used to prepare the water going on the filter by removing settleable matter which would otherwise clog the surface of the filter. The filter removes suspended matter and bacteria and alters the colloidal and dissolved constituents of the water to such an extent as to render it potable. The purpose of the clear-water storage basin is to care for fluctuations in demand during the day or season, in order that the filters may operate at a constant rate.

540. Results of Slow Sand Filtration. The principal purpose of a slow sand filter is to remove bacteria from water. Slow sand filters are highly efficient in this accomplishment, being reliable, when not overloaded, for the removal of 98 to 99 per cent of the bacteria in the raw water. They will also remove suspended and settleable matter not removed in the sedimentation basin, and they will have some beneficial effect on odors and tastes, particularly when due to algae or suspended matter. It is possible that they may, under favorable circumstances, have some slight effect in the reduction of color. They are not highly efficient in the removal of colloidal matter.

Although the removal of 98 to 99 per cent of the bacterial content of a raw water is desirable, it may be insufficient to prevent the passage of pathogenic bacteria so as to assure complete safety against water-borne disease. Absolute safety can be secured by filtration followed by sterilization.

541. Details of a Slow Sand Filter. A slow sand filter consists of a watertight basin containing a layer of sand 3 to 5 ft. thick, supported on a layer of gravel of 6 to 12 in. thick, somewhat as illustrated in Fig. 231. The gravel is underlaid by a system of open-joint underdrains, placed 10 to 20 ft. apart on centers, which lead the water to a single point of outlet where a device is located to control the rate of flow through the filter.

The size of the sand is specified in terms of its effective size and its uniformity coefficient, as defined in Sec. 74. In general, an effective size of 0.35 mm. and a uniformity coefficient of about 1.75 have been found to give satisfaction in slow sand filters.

In the operation of a new sand filter the entire filter is filled with water to a depth of 3 to 5 ft. above the surface of the sand, as indicated in Fig.

231. The water is filtered through the sand at a rate of about 2.5 to 7.5 million gal. per day per acre of filter surface, averaging about 3.0 million gal. per day per acre in most plants. This rate is continued until the difference between the water level on the filter and in the outlet chamber is slightly less than the depth of water above the sand. This difference between the water levels is known as the "loss of head." When it has reached its permissible limit, the filter is thrown out of service, about an inch of sand is scraped from the top of the bed, and the filter is put back into service. The loss of head before cleaning is seldom allowed to exceed the depth of water over the surface of the sand, and after cleaning it may be expected to be about 6 in. A normal period of operation between cleanings should be 2 to 3 months or longer.

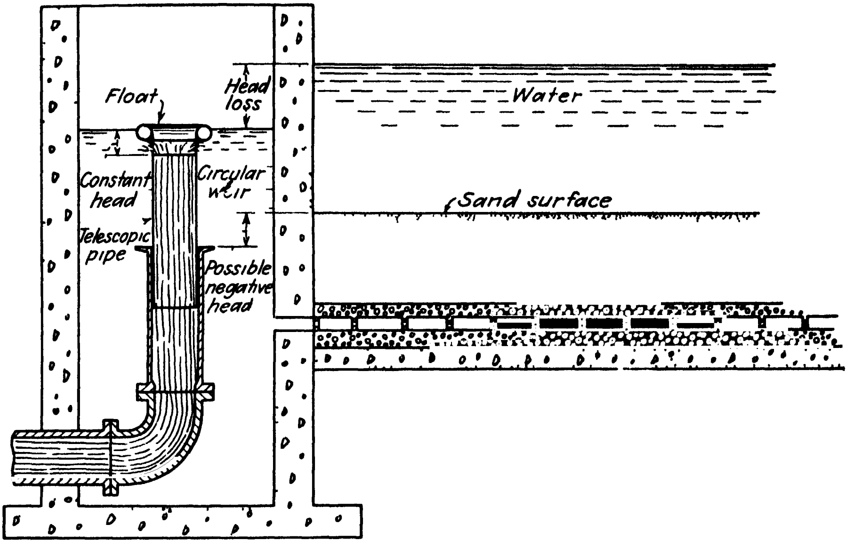

Fig. 231.—Diagram to show possibility of development of negative head in a slow sand filter.

542. The Parts of a Rapid Sand Filter. The parts of a rapid sand filter of the gravity type, stated in the order in which the water passes through them, are as follows:

1. The intake and raw-water pump or gravity conduit.
2. The plain sedimentation basin, if one is used.
3. The mixing basin or other devices for conditioning the water.
4. The coagulating basin in which the coagulant is allowed to precipitate.
5. The filter.
6. The clear-water basin.

The path of the water through a typical rapid sand filter plant is shown diagrammatically in Fig. 232. A general plan of a rapid sand filter plant is illustrated in Fig. 233.

Other essential parts of a plant include the chemical handling and storage equipment; the laboratory; the control building, or head house, and the operating gallery; the wash-water and wash-air equipment; the pipe gallery, the pumping and blowing equipment; and the various appurtenances in connection with the operation of this equipment. The chemical equipment includes the room for receiving and storing the chemicals, the solution storage tanks, and the mixing and feed devices. Solution storage tanks are not required when dry-feed apparatus is provided.

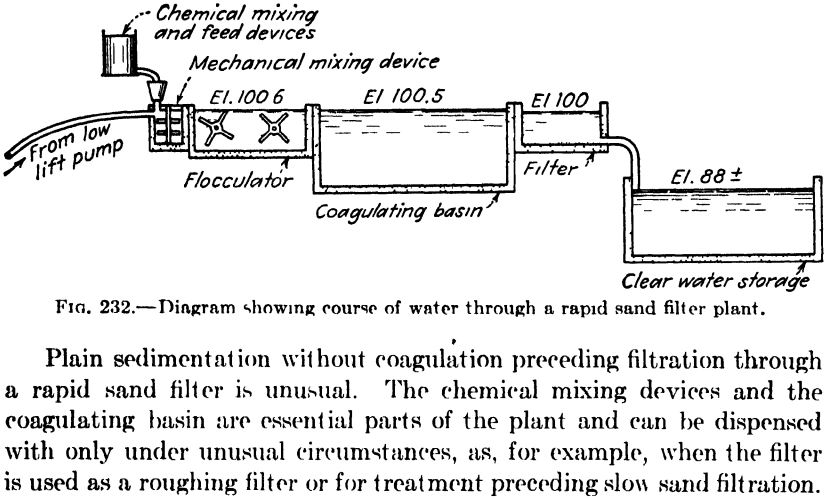

FIG. 232.—Diagram showing course of water through a rapid sand filter plant.

Plain sedimentation without coagulation preceding filtration through a rapid sand filter is unusual. The chemical mixing devices and the coagulating basin are essential parts of the plant and can be dispensed with only under unusual circumstances, as, for example, when the filter is used as a roughing filter or for treatment preceding slow sand filtration. The use of a coagulant to form artificial *schmutzdecke* is essential to the operation of a rapid sand filter. The treatment of water preliminary to filtration is receiving much attention in the design and operation of rapid sand filter plants.[1]

543. Design of a Rapid Sand Filter Plant. The design of a rapid sand filter plant involves the exercise of judgment based on an investigation of local conditions and the amount of purification desired. Such an investigation may involve the operation of a testing plant to study the effect of various forms of water treatment. Before intelligent design can be begun, an investigation should be made upon which the decision to construct a plant, and some details of design, are based.

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[1] See also DIXON, G. G., "New Water Plant Stresses Pretreatment," *Eng. News-Record*, Mar. 30, 1933, p. 395.

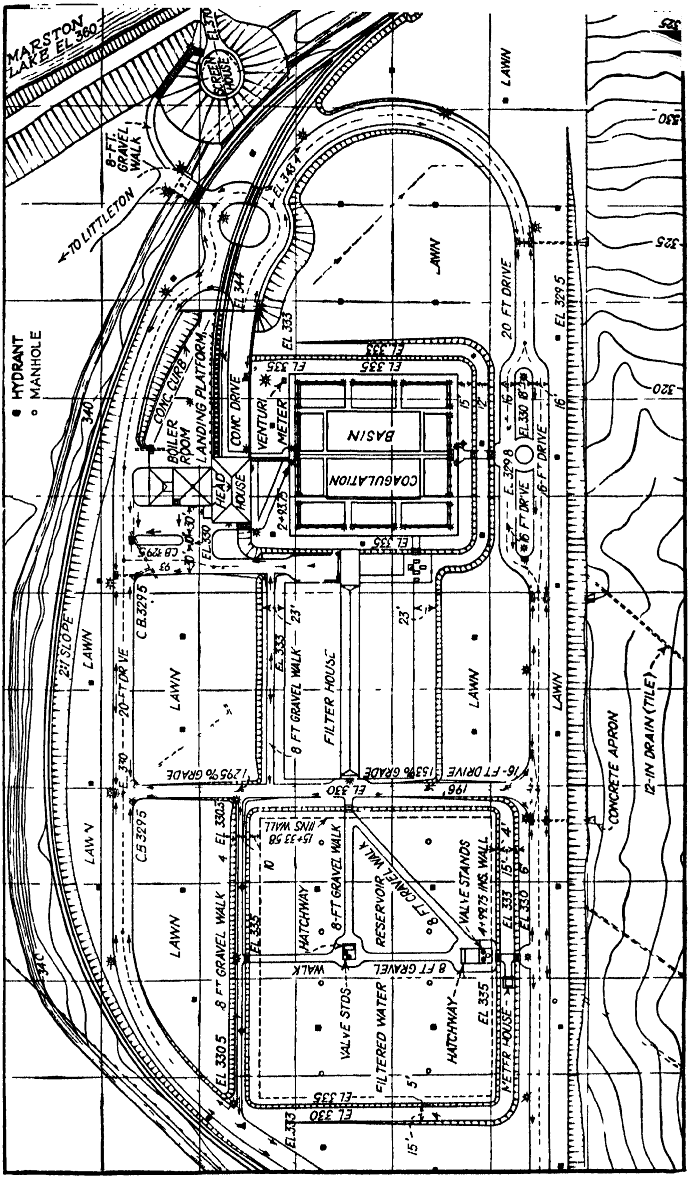

FIG. 233.—General plan of rapid sand filtration, Denver, Colo.

After the capacity of the plant has been determined and the site has been selected, the first step is usually the sketching of the general plan. Although these sketches may be among the first drawings made, the finished general plan may be the last drawing completed because of changes made in the component parts during the design. The general plan of the plant at Denver, Colo., is shown in Fig. 233. It is to be noted that the plan shows the topography and other surface features of the site. Sketches are made of the parts of the plant that are to occupy the site, or pieces of paper are cut out to scale to represent these parts. They are then moved about on the plan until a suitable arrangement has been found.

Features to be considered in the layout of a rapid sand filter plant include the following:

1. Economy in construction and in operation.
2. Future extensions.
3. Foundation conditions.
4. Interior and exterior appearance.
5. Flexibility in operation.
6. Compactness and convenience to minimize materials required, minimize head losses, and simplify operation.
7. Utilization of topography to minimize excavation and backfilling, and to make use of gravity in operation.

A summary of design and operating data of 56 rapid sand filter plants in the United States and Canada is presented by E. A. Hardin.[1]

544. Plans and Specifications. Plans and specifications may be either[2] (1) briefly descriptive, indicating the type of plant required, its capacity, location, etc.; or (2) detailed and complete, covering in minute detail the quality and quantity of materials to be included, the quality of workmanship required, and the exact type and character of every item. The latter type of plan and specifications is in more general use. It is the type prepared by a competent engineer in the best interests of his client to secure a satisfactory plant at the least possible cost.

545. Capacity of Plant. The capacity of a rapid sand filter plant depends on the average rate of demand, the fluctuations in the rate of demand, and the clear-water storage provided. It may be as great as the maximum peak demand load, where no clear-water storage is provided, or as small as the average rate of demand, where the clear-water storage is sufficient to care for all fluctuations in demand. The economical filter capacities and the amount of clear-water storage to be provided lie between these two extremes. The economical capacities can be determined only by a study of the costs of various complete designs using different capacities.

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[1] *Jour. A.W.W.A.*, December, 1942, p. 1847.

[2] STANLEY, W. E., "Specifications for Filter Construction," *Jour. A.W.W.A.*, 1933, p. 105.

The ratio of plant capacity to average pumpage will differ greatly among cities.[1] The ratio appears to be generally more than 2, and it has been found to be as high as 3.2. Large cities will as a rule, have a smaller ratio than smaller cities. In very small plants it may be economical, on account of operating conditions, to operate the plant but a few hours a day, thus necessitating considerable overcapacity of filter plant and reserve storage capacity. Consideration must also be given to safety in the event of partial shutdown, fire reserve, and other local conditions that may affect the plant operation.

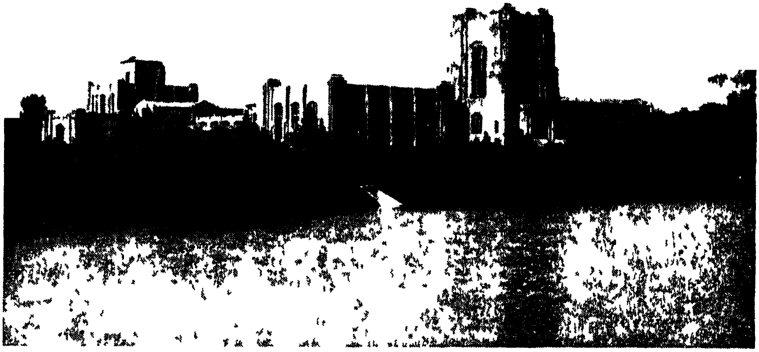

FIG 234 —Rapid sand filter plant, Fort Wayne, Ind. (*Courtesy of L R Mathews*)

546. Filter Buildings. The filter building should be of pleasing design and should be surrounded by attractive grounds. The exterior of the building at Fort Wayne, Ind., is shown in Fig. 234. The architecture is pleasing, and the grounds are attractive. Such structures are required to increase the confidence of the public in the quality of the water delivered by the filter plant and in the ability of the waterworks officials. The public, not acquainted with the technicalities of water purification, is likely to judge the quality of the water as much from the appearance of the plant, both inside and out, as from the appearance and taste of the water. It will be difficult to convince the public that excellent water can be delivered from filters in wooden tubs located within an unsightly shed and surrounded by belching smokestacks and piles of rubbish. Service to the public is of value. This service can be rendered more easily and the public support better assured by the erection of an attractive structure.

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[1] MORRILL, A. B, "The Economic Capacity of Filters and Filtered Water Storage," *Jour. A.W.W.A.*, Vol 16, p. 582, 1926.

The interior of a filter house should include the following:

Laboratory.
Office.
Chemical storage room.
Room for the chemical mixing and feed devices.
Operating gallery.
Filters.
Pump and blower room and other special equipment.

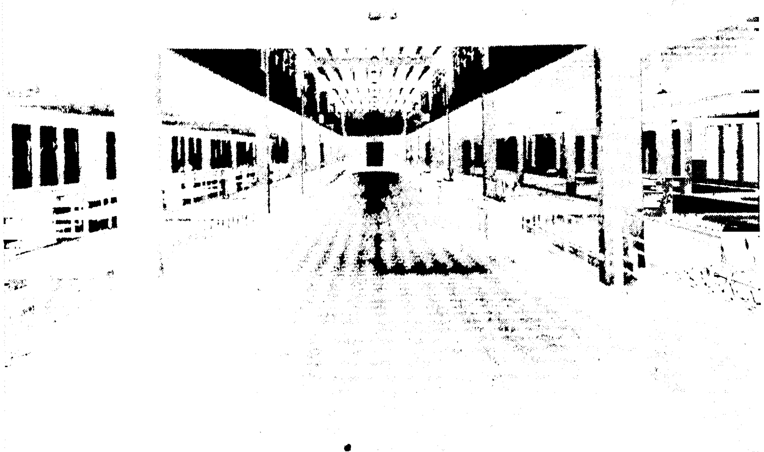

FIG 235 -Interior of filter building at Hammond, Ind (*Courtesy of Paul Hansen*)

The interior of the filter house at Hammond, Ind., is shown in Fig. 235. This plant has been arranged for the convenience of the operator; each room needing outside light is well illuminated; the chemical storage room is large, well ventilated, and dry; the chemicals can be brought into the building with ease; and the interior of the building is pleasing to the eye. White is desirable for interior decoration, as it impresses the visitor favorably and improves the morale of the operating personnel.

The T-shaped plan for a filter building is satisfactory where local conditions will permit. The front portion of two to three stories contains the laboratory, offices, chemical-storage and preparation rooms, and the pump room, if one is required. The shaft of the T is one or two stories and contains the pipe gallery, the operating floor, and the filters. This portion of the building does not always extend to full height over the entire length of the filter. The outside walls of the second story of the building may be supported about one-third to one-half way from the operating gallery to the back of the filter at such a distance that the operator, standing on the floor of the operating gallery, can see the edge of the wall to the water surface at the back of the filter. This is illustrated in Fig. 236, which shows a cross section of the plant at Meriden, Conn. Expense is saved by including only a portion of the filter within the building. The portion of the filters outside the building is covered with a flat concrete slab, which serves to protect the filters from pollution and also as a place to store sand temporarily removed from the filter. To provide for this desirable feature, generous openings, covered with substantial iron covers made tight against rain water, should be provided in the concrete cover.

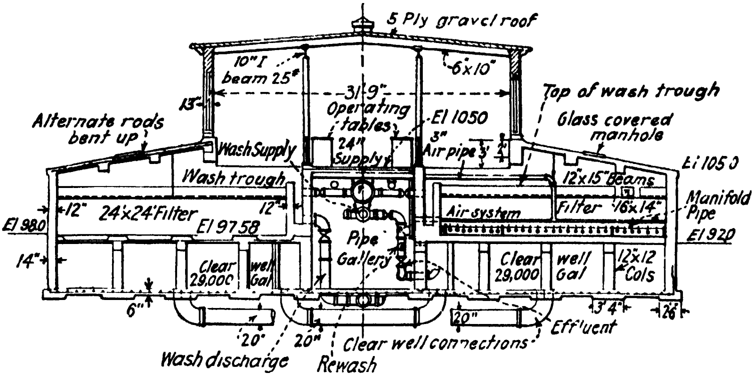

Fig. 236.—Section through rapid sand filtration plant at Meriden, Conn. (*From Water Works, January*, 1927, *p.* 508.)

Excellent interior lighting is important for the sake of appearance and to promote cleanliness. It is essential over the filter to permit proper operation. In some recent plants, skylights over the filter units have greatly improved the daytime illumination. Condensation on the interior walls of the filter building may be minimized by avoiding air movements over the surface of the filters and by the installation of insulation at the roof. The cost of the insulation may be more than repaid by the saving of fuel. Unit heaters in filter plants are conducive to condensation of water and should be avoided.

Filter buildings require more heat than is needed by most buildings because of the high heat losses within the building, through the outside walls and roof, and absorbed by the water being treated. It is essential that the temperature be kept above freezing, particularly around small pipes controlling the hydraulic valves and other equipment. Comfortable temperatures must, of course, be maintained in the laboratory, office, and operator's rooms.

Chemical handling, storage, and feed devices, the laboratory, and other features in connection with the chemical control of the plant should be placed in close proximity to each other within the filter building, both for the convenience of the operator and to assure uninterrupted operation at lower costs. The floor plan of a chemical house at Defiance, Ohio, which embodies these features, is illustrated in Fig. 237.

It is desirable to include a "sight" well by which visitors may observe the clarity of the purified water by seeing an illuminated scale submerged, vertically, in a well lined with glazed white tile. If feasible, the raw water and the clear water may be passed through contiguous wells to emphasize before-and-after effects.

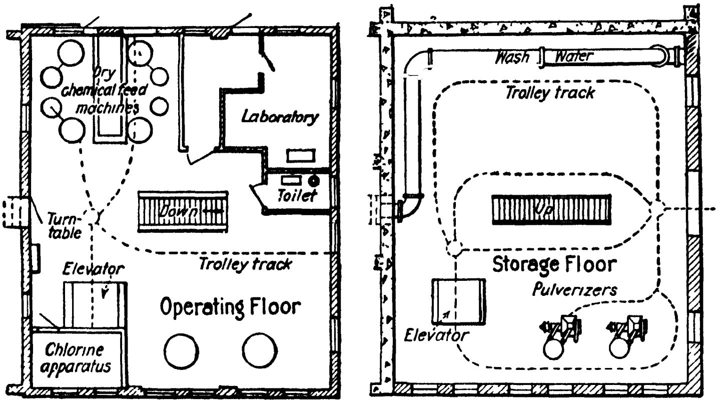

FIG. 237. Floor plan of chemical house at Defiance, Ohio. (*From Jour. A.W.W.A., Vol. 7, p. 145.*)

547. The Filter. Rapid sand filters of the gravity type, of recent design, consist of a rectangular box made of concrete, containing a system of underdrains placed on the bottom of the box. The underdrains are covered with a layer of gravel on top of which a layer of sand, *i.e.*, the filtering material, is placed. Some distance above the surface of the sand are one or more troughs used to take away the dirty water resulting from the washing of the filtering material.

A perspective drawing of a rapid sand filter unit is shown in Fig. 238. Among the important parts of a rapid sand filter unit which are to be determined in design are the following:

1. The capacity of the filter unit.
2. The length and width of the filter unit.
3. The arrangement and dimensions of the wash-water gutters.
4. The filtering material.
5. The underdrainage system.
6. The depth of the filter box.

548. Rate of Filtration. The standard rate of filtration through rapid sand filters, which has been adopted as a result of experience, is 125 million gal. per day per acre of filter surface. This is equivalent to the filtering of 2 g.p.m. per sq. ft. or to the passing of a depth of 384 ft. of water daily through the filter. Practice is tending toward higher rates of filtration in conjunction with the use of coarser filtering material. At Detroit a nominal rate of 180 m.g.d. per acre was adopted in the design of the filters. At Chicago[1] rates of 250 m.g.d. per acre are being used successfully, the adoption of such rates being made possible by the type of preliminary treatment and the filtering sand employed.

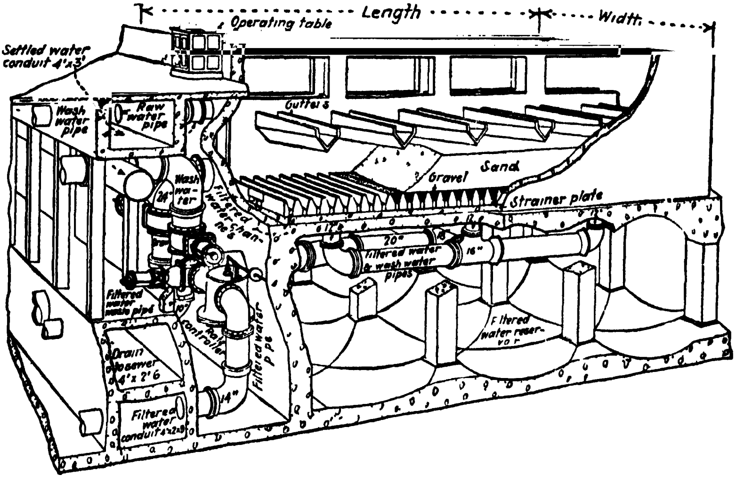

FIG. 238 Perspective drawing of a 2,000,000-gal concrete rapid sand filter unit Inside dimensions 36 ft by 23 ft 6 in Sand area, 738 sq ft. (*Adapted from installation at Evanston, Ill, February,* 1914 *Illinois State Water Survey*)

Baylis[2] has made an extended study of the relation between the rate of filtration and the time the filter runs between washings and has formulated it as

$$H = \frac{K}{M^{1.5}}$$

where H = hours of filter operation
K = a constant depending on clogging tendency of the water
M = rate of filtration, gal. per sq. ft per min.

Values of K, computed from Baylis's report,[2] varied between 52 and 90.

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[1] "High-rate Water Filtration Meeting Practical Tests at Chicago," *Eng. News-Record,* July 10, 1947, p 110.

[2] BAYLIS, J. R, "Experiences in Filtration," *Jour A W W A*, July, 1937, pp. 1010 and 1044.

The initial loss of head increases also as the rate of filtration increases, and there is slight depreciation in the quality of the effluent, "though not enough to be serious."

549. Capacity of Filter Unit. Rapid sand filter units, of the gravity type, can be built of any convenient nominal capacity, but it has been general practice to construct them in some even multiple or fraction of 1 m.g.d. capacity. Units with capacities of 5 m.g.d. are among the largest that have been constructed.

No filter plant handling water for domestic consumption should have less than three units, in order that when one unit is out of service the others may carry on without dangerous overload. The smaller the number of units, the fewer the appurtenances but the larger the wash-water equipment that will be required.

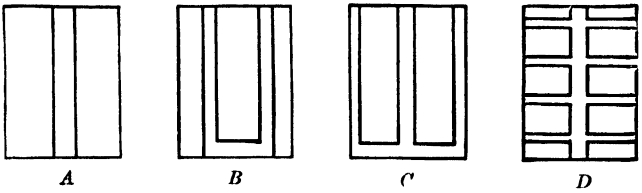

FIG. 239. Diagrams showing arrangements of wash-water gutters in rapid sand filters.

550. Length and Width of the Filter Unit. It is customary to make the length of rectangular filter boxes greater than the width because of greater economy in the utilization of space and in the layout of the piping. Economy and convenience are the only two criteria that fix the relation between these two dimensions; these criteria, in turn, depend on local conditions and the layout of the plant. As a guide to general practice it will be found on investigation of existing plants that where filters are located on both sides of a pipe gallery the ratio of length to width of a filter box lies between 1.66 and 1.11, averaging about 1.25 to 1.33.

551. Wash-water Gutters. Materials used for wash-water gutters include concrete, wrought iron or steel, cast iron, aluminum, monel metal, and occasionally wood.

Various arrangements of wash-water gutters are shown in Fig. 239. It is a general rule of good practice that the horizontal travel of dirty water being washed from the filter shall preferably be less than 3 ft. before reaching a gutter. For this reason the distance from edge to edge of wash-water gutters should not exceed 6 ft., and preferably less than $4\frac{1}{2}$ ft. In Fig. 239*A* the arrangement is suitable only for the smallest filters, which are not over 6 ft. in width. Figure 239*B* shows an arrangement suitable for somewhat larger filters. Figure 239*C* is a modification of this layout in which some of the wash-water gutters are placed against the side walls of the filter. This is not considered good practice because of the loss of the use of one side of the wash-water gutter which is placed against the filter wall. The arrangement in Fig. 239*D* is suitable for and is used in the large filters. It consists of a central gutter extending through the length and the entire depth of the filter, separating it into two almost independent parts. Each of the lateral gutters empties into this deep central gutter. No water is taken over the edge of the central gutter.

The upper edge of the wash-water gutter should be placed sufficiently near to the surface of the sand so that a large quantity of dirty water is not left in the filter after the completion of washing. At the same time the edge of the wash-water gutter should be placed sufficiently far above the surface of the sand so that sand will not be washed into the gutter.

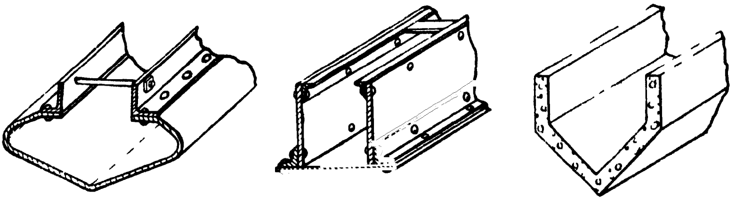

FIG. 240 Types of wash-water gutters.

The edge of the trough should be slightly above the highest elevation of the sand as expanded in washing. Where this height cannot be determined by test, a convenient rule is to place the edge of the gutter as far above the surface of the undisturbed sand surface as the wash-water rises in 1 min. Air and water should not be applied simultaneously with such gutter heights. The gutter should be large enough to carry all the water delivered to it with at least 3 in. between the surface of the water flowing in the gutter and the upper edge of the gutter. Otherwise the gutter becomes submerged, reducing the efficacy of the wash. The gutter may be made with the same cross section throughout its length, or it may be constructed with a varying cross section, increasing in size toward the outlet end. The bottom of the gutter should clear the top of the sand by 2 in. or more. It is not always economical to construct gutters with uniform cross section, because saving may be possible when the bottom of the gutter is sloped so that the front portion is deeper than the rear. Sections of wash-water gutters are shown in Figs. 238 and 240.

In the determination of the dimensions of wash-water gutters, various methods involving certain assumptions of the hydraulics of the gutters are in use. A satisfactory method is illustrated in Fig. 241. Another method for the design of a gutter is presented by Stein.[1]

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[1] *Jour A W W A*, Vol 13, p 411, 1925.

552. Quality of Filtering Materials. Sand is used extensively as a filtering material because of its availability, its relatively low cost, and the satisfactory experience that it has given. Other filtering materials that have been used at a few plants include crushed anthracite, glass, and proprietary products. Tentative standard specifications for filtering materials have been published.[1] The principal advantages claimed for anthracite coal over sand are that lower backwashing velocity is required because of the lower specific gravity of coal and the greater porosity of coal gives more effective filtration. Tests made by Baylis[2] to compare

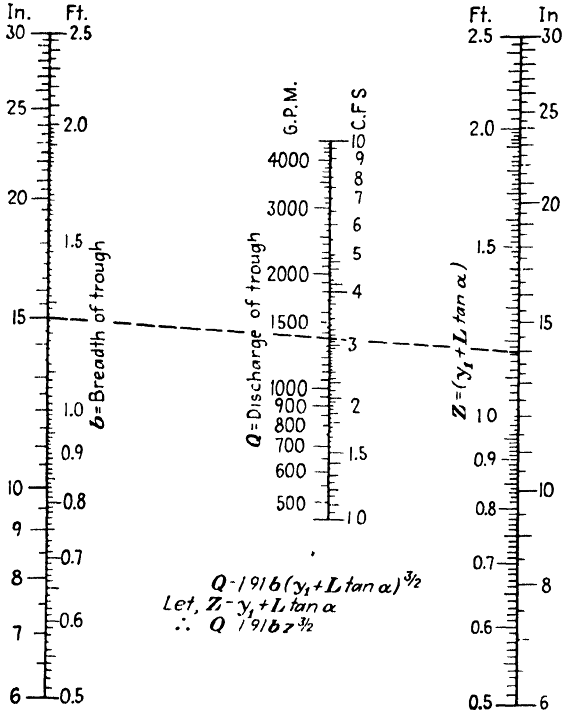

FIG. 241.— Diagram for the design of wash-water gutters. (*From Eng. News-Record, Vol.* 90, *p.* 882, 1923.)

To use this diagram for the design of a wash-water trough, the required capacity and the width of the trough must be known. If the trough is not rectangular in cross section, a trough of equal rectangular area must be computed, and this later changed to the desired shape. With the discharge from the trough Q and the breadth of the trough b known, determine z from the diagram. The depth of the water at the upper end y_1 is next determined from the expression $y_1 = z - LS$, in which L is the length of the trough and S is the slope of the bottom of the trough. The depth of water at the lower end of the trough y_2 is equal to $\frac{2}{3}(y_1 + LS)$.

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[1] *Journal American Water Works Association*, April, 1943, p. 457.

[2] BAYLIS, J. R., "A Study of Filtering Materials for Rapid Sand Filters," *Water Works & Sewerage*, October, 1934, p. 352.

the effectiveness of the various materials showed marked increase in filter runs resulting from the use of sand substitutes.[1]

553. Size and Depth of Filtering Material. The filtering material in a rapid sand filter usually consists of 24 to 30 in. of sand. Since little or no filtering action penetrates as much as 6 in. into a sand bed, practice is tending toward the use of thinner beds than 30 in. In general, the sand should be of such a size as to hold a large quantity of floc without permitting its passage through the filter and without increasing the difficulties of washing; it should permit long filter runs; and when washed it should cleanse itself so as to be free from adhering particles without requiring a velocity of wash water so high as to carry away particles of sand. Some of these requirements are mutually inconsistent. For example, fine sand will prevent the passage of floc and will deliver a good quality of effluent but will reduce the length of the filter run and may make washing difficult, whereas coarse sand will do the reverse. A sand layer made up of small-size grains will expand further when being washed than one consisting of large-size grains subjected to the same rate of washing. As the finer grains become more widely separated, their tendency to strike each other is lessened, and the difficulty of cleansing is increased. On the other hand, the larger the sand grains, the greater the force with which they will strike together and the easier it will be to keep the bed clean. With coarser sand, filters can be run longer between washings, they can be washed cleaner, the number and size of mud deposits will be greatly reduced, and it will not be necessary to remove and wash the sand as often. However, if sand becomes too coarse the efficiency of bacterial removal diminishes. Filter beds of coarse sand should be deeper than those of fine sand, and the rate of rise of wash water should be greater for equal effects in filter washing. The designer must decide on the most advantageous combination of conditions and select the filtering material accordingly.

The effective size of sand (Hazen's) ordinarily used is between 0.40 and 0.55 mm., with a uniformity coefficient not greater than 1.8.[2] Variations are found in practice between 0.3 and 0.6 mm., with a tendency toward the adoption of larger sizes. Successful application of filtration rates double the standard rate. At Chicago[1] successful use of greatly increased filtration rates is attributed, in part, to the specifications of filter sand. It has been shown by Chicago experience that the expression of size in terms of Hazen's effective size is not suited to the high rates in use there. The term *per cent size* is used, that being the size of particle that has the stated per cent, by weight, of materials finer in size. Chicago's filter-sand specifications and a typical size distribution of samples of sand are shown in Fig. 242. Uniformity of size is desirable because of the tendency of sand grains to group in strata of similar sizes due to the action of the wash water. A stratum of fine material beneath a stratum of coarse material may cause clogging within the filter, and a stratum of coarse material beneath a stratum of fine material may result in a reduction of pressure within the filter sufficient to release dissolved gases, causing either "air binding" or a "blow" that will break through the filter surface.

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[1] Turner and Scott, "Accounting for Anthracite's Superiority to Sand as a Filter Medium," *Water Works & Sewerage*, April, 1933, p 135, and September, 1933, p 330; Hoover, C. P., "Experience with Anthracite Coal," *Water Works & Sewerage*, November, 1933, p 394, Turner and Young, "Attrition Loss of Anthracite," *Water Works & Sewerage*, July, 1933, p. 346; Ripple, O. J., "Experiences with Anthracite as a Filtering Material," *Water Works & Sewerage*, Vol. 83, p 255, 1936; Turner, H G., "Anthrafilt and Its Significance," *Jour. A.W W.A.*, April, 1944, p. 431.

[2] Committee Report, "Filter Sand for Water Purification Practice," *Proc. A.S.C.E.*, Vol. 62, p 1543, 1936.

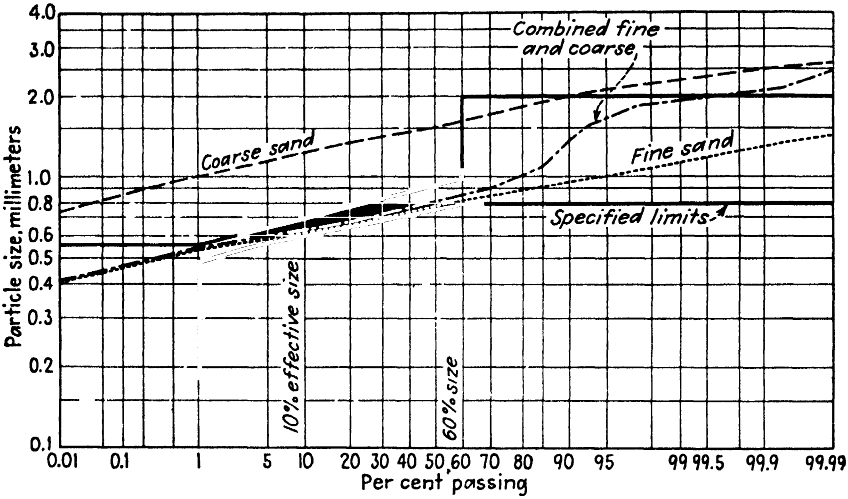

FIG. 242. Graph showing Chicago's filter sand specifications and a typical size distribution of samples of sand obtained.

554. Filter Gravel. Gravel is placed between the sand and the underdrainage system (1) to prevent sand from entering the underdrains and (2) to aid in uniform distribution of the wash water. The size and depth of the gravel should accomplish both purposes without being displaced by the rising wash water. The gravel is usually graded from coarser sizes at the bottom to finer sizes at the top. Sizes in use vary from $1\frac{1}{2}$ in. at the bottom to $\frac{1}{8}$ in. at the top of the gravel layer,[2] with depths of gravel between 9 and 18 in. The faster the rate of application of wash water, the larger the gravel size required and the deeper the bed. In some plants[1] metal screens have been fastened between the gravel and the sand to prevent movement of the gravel.

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[1] See "High-rate Water Filtration Meeting Practical Tests at Chicago," *Eng. News-Record*, July 10, 1947, p. 110.

[2] See also BARKER, S. T., and H. F. FERGUSON, "Water Purification Plant Design and Operation," *Jour. A.W.W.A.*, Vol. 13, pp. 150 and 154, 1925.

555. Underdrainage Systems.[2] Filter underdrains are used also to distribute the wash water uniformly beneath the bed. They should be strong enough to withstand the sudden vibrations of water pressure put upon them. The best distribution of wash water is obtained when the loss of head in the distributing pipes is relatively low in comparison with the loss through the strainer or pipe perforations. Types of underdrains in use include (1) perforated pipes, (2) pipe and strainers, (3) ridge and valley, and (4) perforated plates or other false bottoms.

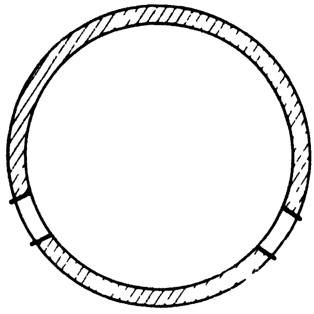

FIG. 243. —Perforated underdrain pipe

556. Perforated-pipe Underdrains.[3] A perforated-pipe underdrain, such as is illustrated in Fig. 243, consists of a central manifold to which are attached a series of branch pipes. Cast iron is most commonly used for manifold and laterals. Holes are bored in the lower portion of the branch pipes, somewhat as indicated in Fig. 243. The holes are located so that the wash water is directed downward at an angle between 30 and 60 deg. with the vertical. Erosion or corrosion of the metal around these holes may be minimized by lining the holes with a brass or bronze bushing. It has been customary to place the branch pipes on 6- to 12-in. centers, the larger distance being preferred, with $\frac{1}{4}$-in. to $\frac{1}{2}$-in.-diameter holes placed on 3- to 8-in. centers along the pipe. The smaller the spacing the smaller the individual holes. In some successful designs there are two or more manifolds interconnected by branch pipes.

As a guide to satisfactory design the following principles, attributed to Jenks and Ellms,[4] may be used:[5]

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[1] See also LABOON, J. F., "New Features of Design to Improve Plant Operation," *Eng. News-Record*, Sept. 13, 1934 p. 336.

[2] A Symposium, "Filter Underdrain, Sand-bed, and Wash-water Experience," *Eng. News-Record*, Vol. 85, pp. 934 and 984, 1920; and Vol. 86, p. 371, 1920.

[3] See also SCHMITT, D. A., and P. O. MACQUEEN, "Perforated Pipe Underdrains for Rapid Sand Filters," *Jour. A.W.W.A.*, June, 1942, p. 857.

[4] See also JENKS, H. N., "Filter Design as Related to Operation," *Jour. A.W.W.A.*, Vol. 28, p. 1541, 1936; and ELLMS, J. W., "Design of Perforated Pipe Strainer Systems," *Jour. A.W.W.A.*, Vol. 18, p. 664, 1927.

[5] See also SCHMITT, E. A., and P. O. MACQUEEN, "Perforated Pipe Underdrains for Rapid Sand Filters," *Jour. A.W.W.A.*, June, 1942, p. 857.

1. The ratio of length of lateral to its diameter should not exceed 60.
2. The diameter of perforations in the laterals should be between ¼ and ½ in.
3. The spacing of the perforations along the lateral may vary from 3 in. for a diameter of perforation of ¼ in., to 8 in. for a diameter of perforations of ½ in.
4. The ratio of the total area of the perforations in the underdrainage system to the total cross-sectional area of the laterals should not exceed 0.5 for a diameter of perforation of ½ in., and should decrease to 0.25 for a diameter of perforation of ¼ in.
5. The ratio of the total area of the perforations in the underdrainage system to the entire filter area may be as low as 0.002.
6. The spacing of the laterals may be as great as 12 in. for satisfactory diffusion but is limited by the total head available.
7. The rate of washing may be varied from 6 to 36 in. per min. provided the foregoing factors are used in design.
8. The sum of the cross-sectional areas of the laterals should be at least twice the sum of the cross-sectional areas of the perforations in the laterals.
9. The area of the manifold feeding the laterals should be 1.75 to 2.00 times the sum of the cross-sectional areas of the laterals which are fed in the washing process.

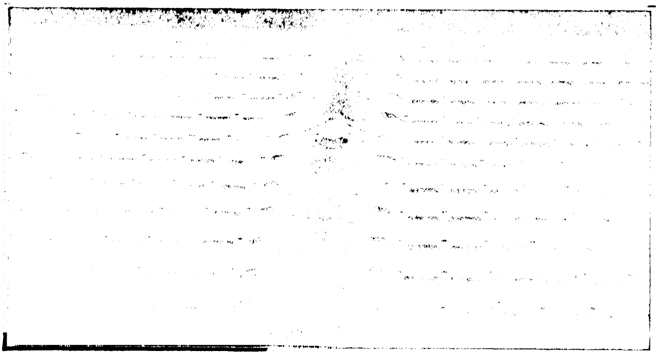

FIG. 244. Pipe-and-strainer system of underdrains. (*Courtesy of American Water Softener Co.*)

The principles upon which the number and size of openings in perforated pipe underdrains are determined are equally applicable to other systems of underdrainage.

557. Pipe-and-strainer Underdrains. This type of underdrainage system is similar to the perforated-pipe system, except that strainers, with small openings or perforations, somewhat as shown in Fig. 245, are inserted into the laterals at convenient distances. This is the first type of underdrain that was used in rapid sand filters. It is now most commonly used for small filters, but it is being superseded to some extent even in that field by perforated pipe underdrains.

This type of underdrainage system consists of a central manifold with laterals on approximately 6-in. centers. Strainers, usually made of brass, are inserted into the top of the laterals, and sometimes into the top of the manifoid, on 6-in. centers. It is desirable, but sometimes difficult, to place all strainers at the same elevation.

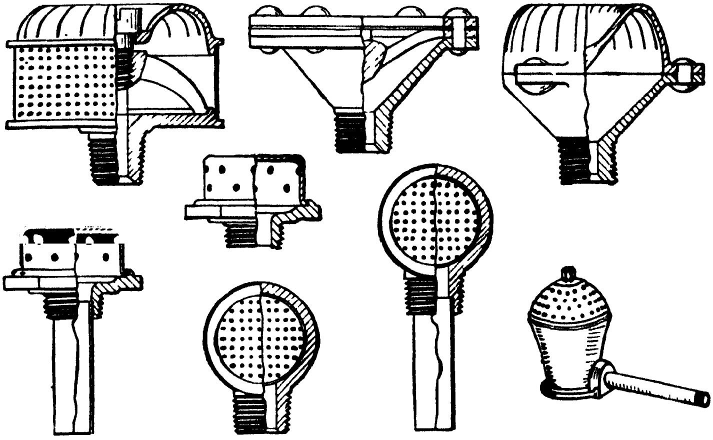

FIG. 245. Types of strainer heads. (*Courtesy of American Water Softener Co.*)

Dead space beneath the strainers may be avoided by filling beneath the strainers with lean easily removable concrete. The openings in the strainers should be small in order to provide good wash-water distribution. For the same reason the cross-sectional area of each lateral should be more than two to four times the area of the holes in the strainers on the lateral, and the area of the manifold should have more than twice the cross-sectional area of the laterals on it.

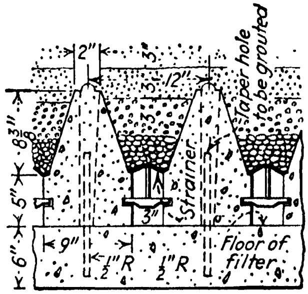

FIG. 246.—Section through concrete ridge-and-valley underdrain at Evanston, Ill.

558. Ridge-and-valley Underdrains. The ridge-and-valley underdrainage system has been used in large filter units because of the smaller expense in connection with its construction and the satisfactory results obtained from it. The system consists of ridges of concrete placed on centers from 8 to 12 in. apart. A perforated brass screen is supported between these ridges about 4 in. above the bottom of the filter. The space under the screen serves as the channel through which the water is to flow, and the perforated brass screen serves as the strainer. The size and number of holes in the strainer plates are determined on the same principles as for the pipe-and-strainer system of underdrainage, and the velocity of flow of wash water is designed similarly.

Care must be taken in fastening the brass strainer plates securely in place, as the pressure of the wash water sometimes displaces them. A successful design is shown in Fig. 246.

559. Miscellaneous Underdrainage Systems. Other types of underdrainage systems that have been used with reported success include the Wheeler filter bottom,[1] an arrangement consisting of conical or pyramidal openings in a concrete bottom, with the point of the cone or pyramid down. No strainer is used, but the pyramidal openings are filled with heavy balls varying in diameter from 3 to 1.2 in. These balls are carefully placed in the pyramid to secure the least voids.

An underdrainage system of wood was used at Sacramento, Calif.,[2] as well as a cement-gravel slab with vitrified-clay pipe underdrainage system.[3] Other systems have been used, as, for example, the false bottom of perforated plates at Little Falls, N.J., the nonmetal system of F. B. Leopold Co.,[4] and a system of porous plates, such as the Aloxite plates manufactured by the Carborundum Co.[5] Experience at Warren, Ohio,[6] where both manifold and strainers and false-bottom underdrains were used favored the false bottom with umbrella-type strainers.

A drainage system placed immediately below the surface of the sand, to supplement the underdrainage system placed at the bottom of the filter bed, was introduced by Ira H. Jewell[7] in 1937. The strainer system consists of a manifold with laterals and outlets placed 12 to 15 in. on centers. The outlets are surrounded by fine screens, placed about 6 in. below the sand surface. In the Kenilworth plant there are 110 screens on a unit of 500,000 gal. per day. The results reported from the operation of the Kenilworth plant show that the device permits an increase in the rate of filtration and greatly improves the washing of the filter.

560. Air Distribution. In filters using air for washing, the strainer system must be adapted to the use of air, or a special air-distributing system must be provided. The pipes and manifold of this system are placed immediately above the pipes of the water underdrainage system in such a manner as to interfere as little as possible with the water strainers. The design of a separate air-distributing system follows principles similar to those followed in the design of the pipe-and-strainer system for water. Perforated strainers are used for the distribution of the air. The holes are made about the same size and of the same total area as the holes for the water system. The size of the air pipes is based on a velocity of flow of the compressed air of about 2,000 f.p.m.

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[1] See also WESTON, R. S., "The Wheeler Filter Bottom," *Eng. News-Record*, Vol. 72, p. 22, 1914.

[2] See also JENKS, H. N., "Experience with Wood Grating in Sacramento Filters," with discussion, *Eng. News-Rec.*, Vol. 94, pp. 631 and 1062. 1925.

[3] See also JENKS, H. N., "A Cemented Gravel Slab Vitrified-clay Pipe Distribution System for Rapid Sand Filters," *Jour. A.W.W.A.*, Vol. 16, p. 542, 1926.

[4] See also HOWARD, N. J., "Water Supply and Purification during 1932," *Water Works & Sewerage*, January, 1933, p. 1.

[5] See also CAMP, T. R., "Porous Plates for Use in Filter Bottoms," *Jour. New Engl. Water Works Assoc.*, March, 1935, p. 10; and *Water Works & Sewerage*, March, 1937, p. 84.

[6] O'CONNOR, P. J., 12*th Ann. Rept., Ohio Conference on Water Purification*, 1932.

[7] JEWELL, IRA H., "Increasing Filter Capacity by Subsurface Filter and Wash Units," *Water Works Eng.*, Feb. 2, 1938, p. 159.

In some unusual cases the same underdrainage system has been used for the distribution of both air and water. Both cannot be forced through the strainers at the same time unless special provision has been made therefor. It may be desirable to use water simultaneously with the air wash. This can be done if the strainer head is arranged with a long stem extending into the lateral and with a small air hole in the stem near the top of the inside of the air-distributing pipe.

561. Depth of Filter Box. Considerations of economy demand that the filter box be made as shallow as possible. The depth may be determined by the required distance between the bottom of the box and the necessary freeboard above the edge of the wash-water gutter, or the depth may be fixed by the arrangement of the effluent pipe and the maximum head to be permitted in the operation of the filter.

The *minimum* depth of the filter box will be the sum of the following dimensions:

1. The height required by the strainer system and the covering of gravel. This is seldom less than 8 in.
2. The thickness of the sand bed. This is quite commonly made about 30 in.
3. The distance between the top of the sand and the edge of the wash-water gutter. This is equal to the distance that the wash water will rise in the filter in 1 min., if wash air is not used.
4. The desirable freeboard above the wash-water gutter. The minimum should be about 6 in., and a much greater depth is preferable to assure good weir action over the edge of the gutter and to provide a factor of safety against the overflowing of the filter box.

Filter boxes are seldom constructed with a depth less than 6 to 8 ft. and are usually deeper.

In fixing the requisite depth of the filter box the greatest permissible head loss in operating the filter is frequently of more importance than the factors listed. Where the pressure in the filter is not permitted to fall below atmospheric, the depth of the filter box must be equal to the maximum loss of head in the filter plus sufficient freeboard to prevent loss of water over the top of the filter. Where the pressure in the filter may be below atmospheric, the depth of the filter box may be equal to the maximum permissible head loss in the filter plus the necessary freeboard and less the allowable negative head at the bottom of the filter. The greater the negative head, the less the necessary depth of the filter box, but the greater the possible difficulties in operation.[1]

562. Negative Head. The negative head at any point in a filter is equal to the intensity of vacuum at that point and is usually a maximum at the point where the water enters the underdrainage system. It is represented by *N* in Fig. 247.

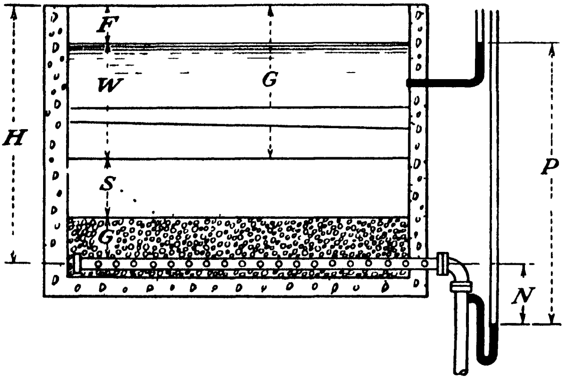

Fig. 247. Diagram illustrating negative head in a rapid sand filter.

An undesirable feature of the existence of negative head in the operation of a filter is the occasional release of dissolved gases from the water into the sand layer, due to the decrease of pressure. These gases may either accumulate in the sand, causing what is known as *air binding*,[2] or they may break through the filter, leaving an opening through which water may pass with imperfect treatment. A result of air binding is the failure of the water to pass through the filter where the bubble of gas has accumulated. This may result in loss of capacity of the filter or an overloading of other portions of the filter. The use of negative head has the advantages, in design, of permitting shallower filter boxes and, in operation, of permitting longer filter runs, provided no other difficulties arise.

563. The Loss of Head in Filter Operation. The frictional resistance to the passage of water through the filtering medium and the underdrains of a rapid sand filter causes a loss of head in the operation of the filter. This loss of head is equal to the vertical distance between the surface of the water on the filter and the elevation of the hydraulic grade line at the filter outlet. This elevation can be determined by placing a piezometer in the effluent pipe and observing the elevation of the water in the piezometer tube.

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[1] See also Morrill, A. B., "The Design and Care of Rapid Sand Filters," *Jour. A.W.W.A.*, April, 1934, p. 445.

[2] See also Baylis, J. R., "Air Binding of Filter Beds," *Water Works & Sewerage*, November, 1930, p. 379.

The loss of head in a rapid sand filter immediately after washing should be in the neighborhood of 4 to 6 in. The finer the sand or the greater the rate of filtration, the greater the initial head loss. As the filter grows dirty during a run, the head loss builds up until the maximum permissible loss of 10 to 15 ft. is reached. The maximum loss of head usually assumed is 8 to 10 ft. Greater losses of head are undesirable because of the tendency of the water pressure to pack the sand so tightly as to cause difficulties in washing or possibly the water to break through the sand without filtration.

564. Control of Water Level on Filter. The elevation of the surface of the water on a rapid sand filter is frequently fixed by the elevation of the outlet weir from the coagulating basin. The water surface in the filter cannot fall below this elevation, for, if it tends to do so, water is fed more rapidly from the coagulating basin to maintain the constant level. The loss of head in the conduits between the coagulating basin and the filter, usually being so slight as to be negligible, makes it possible to design the plant with the water surface on the filter and on the coagulating basin at the same elevation. Where water is to be pumped on to the surface of the filter, or other method of feeding the filter is used, the water level on the filter may be maintained by use of a float-controlled inlet valve which opens as the water level tends to sink and closes as it tends to rise. One method of rate control requires a constant supply of water to the filter, resulting in an increase of the depth of water on the filter as the loss of head increases during operation.

565. Rate Control. A constant rate of filtration is desirable on a rapid sand filter, because either a sudden increase in rate may cause water to break through the filtering material without proper treatment, or the sudden increase in negative pressure required to pull water through the filter may release dissolved gases. A sudden reduction in the rate of filtration may likewise release a bubble of gas that has been entrained in the sand, causing it to make a hole through the filter bed. A constant rate of filtration may be secured by hand or by automatic control.

566. Manual Rate Control. There are two successful methods for manual rate control. In the first one, the water is applied to the filter at a constant rate. The constant rate of applied water is secured through the control of the influent valve. At the start of a filter run, the effluent valve is opened slightly, so that the loss of head through it and through the sand may be such as to maintain just sufficient depth of water on the sand to prevent the surface of the sand from disturbance by the entering water. As the sand becomes clogged, and the loss of head through the filter increases, the depth of water on the sand will increase. After it has increased to any desired limit, the effluent valve should be opened in order to decrease the loss of head through it. When the effluent valve is finally opened wide and the surface of the water approaches the top of the filter box, it is necessary to wash the filter for another run. It is to be noted that in this method of manual control the elevation of the surface of water in the filter is constantly changing.

In another method of manual control, a constant depth of water can be maintained on the filter by means of a float-controlled influent valve. The effluent will pass through the venturi meter or other calibrated tube or orifice. The rate of discharge can be maintained approximately constant by opening the discharge valve to compensate for the loss of head through the filter. Such a method is not used frequently and is suitable mainly in small temporary installations.

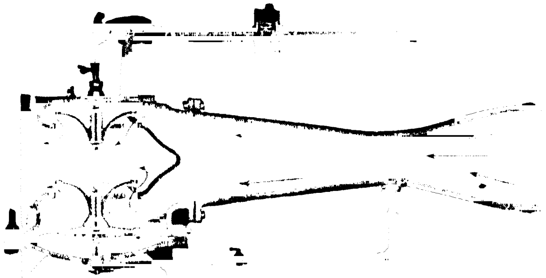

FIG. 248 Simplex rate controller (*Courtesy of Simplex Valve and Meter Co.*)

567. Automatic Rate Control. There are two types of automatic rate controller, one open and the other submerged. An open rate controller depends for its actions on a float attached to a weir within the controller. The depth of water on the weir is held constant through the rise and fall of the float. Since the float in the open rate controller must rest on the surface of the water exposed to atmospheric pressure, the top of the controller must be open to the atmosphere. Open-type rate controllers are seldom used today on rapid sand filters. A submerged type is preferable, because it permits any pressure desirable in the effluent pipe.

Many types of submerged rate controller are in use, but practically all of them depend on the priciple of the Venturi tube for their action. Rate controllers are manufactured devices whose dimensions and capacities have not been generally standardized. The Simplex rate controller is illustrated in Fig. 248. It is a commonly used type. It operates as follows: The direction of flow of water through the apparatus is shown in the figure. If the rate of flow increases through the Venturi tube, the pressure in the control chamber where the balanced valve is located and the pressure at the throat will both increase, but the increase in the control chamber will be the larger. The increased pressure on top of the flexible disk shown in the figure will force the balanced valve down, thus cutting down the rate of flow. The position of the balanced valve fixes the rate of flow through the device, and this position is regulated by means of the movable weight on the lever, as shown.

When the clear-water basin of a filtration plant becomes nearly filled, it is desirable to cut down the rate of filtration. This may be done automatically by a master controller which affects the individual controller on each filter. In some plants, rate controllers or master controllers are so set that the filters will operate at various rates according to the depth of water in the clear-water basin. Such installations are not common, however.

The accuracy of operation of a rate controller can be tested by filling the filter box with water, closing all but the effluent valve, and observing the rate at which the water level drops in the filter. The discrepancy between the setting of the rate controller and the rate of the dropping of the water in the filter box will indicate the error of the controller.

568. Pipe Gallery. In a rapid sand filter plant containing a large number of filter units, it is customary to place the filters in rows with the piping or other conduits between them. This space is called the pipe gallery.

All the conduits conveying water or air to or from the filters should be placed in an accessible position in the pipe gallery. The conduits commonly to be found in a pipe gallery include the influent; the effluent; the wash water; the waste water, or sewer; and the wash air, if wash air is used. Other pipes, such as heating pipes and electrical conduits, may also be placed in the pipe gallery for convenience. The purpose of each of the conduits in the preceding list is evident from its name. Another pipe which is installed in most filter plants is the rewash, or filtered-water, waste pipe. This serves to connect the effluent from the filter directly to the sewer. It may be used to drain the filter into the sewer, or after a filter has been washed any untreated water remaining in the filter can be wasted into the sewer.

The sizes of the conduits in the pipe gallery are dependent on the quantities of water to be carried in them and on the desired velocities of flow. The velocity of flow in the influent pipe is usually made slower than would be called for by considerations of economy alone, because of a desire to avoid breaking up of the floc. Velocities of 1 to 2 f.p.s. have been found satisfactory in the influent conduit. The velocity of flow in the effluent conduit is controlled by considerations of convenience and economy, as a result of which somewhat higher velocities are used. Where cast-iron effluent pipes are used, a velocity of about 4 f.p.s. is economical. Concrete conduits would call for slightly lower velocities. If the fall between the filter and the clear-water basin is large enough to permit higher velocities in the effluent pipe, all the available head should be consumed, if convenient.

The size of the wash-water pipe is fixed by considerations of economy and convenience. Since this conduit is used only intermittently and for short periods of time, the velocity is higher than in the effluent conduit. A velocity of about 10 f.p.s. is economical for cast-iron wash-water pipes. Concrete is seldom used for wash-water conduit because of the necessary pressure at which the water must be delivered. A velocity of flow of not less than 30 f.p.s. will be found economical and satisfactory in the wash-air pipe. The velocity of flow in the sewer is fixed by the available head, the design being based on the principles of hydraulics and Kutter's, or similar, formulas. As the heads available for the branch sewers are usually quite large, the size of the branch sewer is usually fixed on an arbitrary assumption of a velocity not greater than 5 to 10 f.p.s.

The size of the main conduits in the gallery should be changed along the gallery as the branch pipes remove or add water to them, except in the case of the wash-water or wash-air pipes. Where only one filter is to be washed at a time, the flow in the branch wash-water or wash-air pipe is the same as the flow in the main pipe, and hence the branch and the main pipes are of the same size.

Some point of the branch air pipe, should be placed at an elevation higher than the highest possible elevation of the surface of the water in the filter. This precaution is taken in order to avoid the possibility of siphoning the contents of the filter box into the air piping.

The location of the pipes and the conduits in the pipe gallery should be such as to secure the greatest convenience in working on and inspecting the conduits. Good light, good ventilation, good drainage, and a wide, unobstructed passageway for the entire length of the gallery are conducive to proper care of the equipment placed in it.

The pipe gallery at Meriden, Conn., embodies some of these features, as shown in Fig. 236. Details of a pipe gallery are shown in Fig. 238.

Cast iron and concrete are the principal materials used for the pipes and conduits in a pipe gallery. Steel pipe is sometimes used for conveying wash air. Three types of joint are used for connecting the cast-iron pipe, as explained in Sec. 299. Threaded joints are seldom used on pipes larger than 8 in. in diameter. Flange joints make a neat appearance, are easy to connect when the pipe is in proper position, and are extensively used. An undesirable feature of the use of flanged joints is the difficulty of taking up small discrepancies in the position of the pipe, as the flange joint is practically inflexible. The bell-and-spigot joint is probably the least expensive to make, and slight errors in the location of the pipe can be taken up by the play allowed in the bell. The joint occupies considerable space, however, and considerable space is required in which to calk it. Specials with bell-and-spigot ends take more space than is required by any other form of cast-iron special. When bell-and-spigot joints are used, care must be taken to prevent the blowing open of the joints due to unbalanced pressure on dead ends. To avoid this possibility, some form of end restraint is necessary on a bell-and-spigot pipe line with lead joints. Where the end of the pipes cannot be anchored or braced against a permanent structure, it may be necessary to tie the pipe together with cables.

The connections, specials, and lengths of pipe used in fitting up the pipe gallery should be made according to standards available in manufacturers' stocks in order to secure the greatest economy. The exact sizes of the various standards can be found in any good handbook and many of the catalogues issued by the manufacturers. The cost to manufacture fittings not in stock is so much greater than for standard fittings that the use of nonstandard fittings in small plants is almost prohibitive.

The pipes of the pipe gallery should be well supported because of their weight and the shocks that come on them in operation. In general, each piece of pipe should be so supported that the weight does not come on a joint where the axis of the pipe is in any position other than vertical. This is particularly true of bell-and-spigot joints.

569. Washing of Filters. A filter should be washed when the loss of head through it has reached the maximum permissible. The washing of a rapid sand filter is accomplished by reversing the flow of water through it. In some plants the sand is agitated with air or by mechanical rakes while it is suspended in the rising stream of wash water. The use of mechanical rakes is now obsolete, and the use of air is not extensive.

The quality of the wash water should be as good as that of the effluent from the filters where filtration is for hygienic purposes. The dirty water from the filters is usually run to waste, but in some plants this water is of sufficient value to justify its reclamation and purification. This may be accomplished by sedimentation, after which the effluent from the wash-water salvage basin is passed through the filters.

The steps in the washing of a rapid sand filter, in which water alone is used for washing, are as follows:

1. Close the influent valve and allow the water in the filter to drain down somewhat. Some operators allow the water to drain down to the edge of the wash-water gutter; others, to near the top of the sand; and others, to a mid-way point.

2. Close the effluent valve.
3. Open the waste-water valve to the sewer.
4. Open the wash-water valve gradually, to prevent dislodgment of the gravel by entrapped air, and allow the wash water to flow through the filter and out through the wash-water gutters until it is certain that the filter is properly cleaned.
5. Close the wash-water valve.
6. Close the waste-water valve to the sewer after the water in the filter has drained down to the edge of the wash-water gutter.
7. Open the influent valve slightly.
8. Open the rewash valve, and allow some of the water to drain to the sewer. (This step is frequently omitted.)
9. Close the rewash valve.
10. Open the effluent and the influent valves. The filter is now back in service.

The wash water, as a rule, should be turned on gently at first to avoid concentrating it at a few spots in the underdrainage system, resulting in mixing the gravel with the sand. At some plants, wash-water rate controllers are used which operate either automatically or manually.[1]

When air agitation is used, the wash-air valve may be opened at the same time that the wash-water valve is opened, or the wash-air and the wash-water valves may be opened alternately. It is usually considered undesirable to apply both air and water simultaneously because of danger of disturbing the gravel or of losing sand into the wash-water gutter. Jet action of wash water against the gravel or against the sand above the gravel is undesirable to avoid dangers of disturbing the gravel or causing mounds and craters in the filter bed. The wash-air valve is sometimes opened before the wash-water valve in order to break up the sand surface, making the passage of the wash water through the sand easier at the start.

570. Surface Wash. Surface wash, in rapid sand filter practice, refers to a method of applying wash water at or near the surface of the sand in the filter. Although not new in waterworks practice,[2] its redevelopment and its increasing use, particularly on filters already in operation, have been due to improvements in design and methods of operation. Many different schemes are used for which satisfaction is claimed.[3] In some plants a wash-water distribution system has been suspended about 4 in. above the undisturbed sand surface with holes facing downward supplying jets under high pressure. During washing both surface and subsurface wash may be used simultaneously, the swirling motion of the wash water serving to scrub the sand grains effectively against one another. In some plants the surface may be broken up with a high pressure stream from a fire hose; in others a surface washing device may be improvised by attaching a length of perforated pipe to a fire hose, the device being moved about on the sand surface during washing.

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[1] See also DODD, C. I., "Improvements in Water Works Equipment and Materials," *Jour. A.W.W.A.*, March, 1937, p. 322.

[2] See also DEBERARD, W. W., and LANGDON PEARSE, "The Testing Station and Filter Plant at Oakland, Calif.," *Eng. Record*, 58, p. 97, 1908.

[3] See also BAYLIS, J. R., "Filter Bed Troubles and Their Elimination," *Jour. New Engl. Water Works Assoc.*, Vol. 51, pp. 1 and 26, 1937.

571. Sand Expansion. The wash-water valve should be opened sufficiently to expand the sand layer about 50 per cent, *i.e.*, to raise the level of the surface of the sand so that the sand layer occupies 1.5 times the volume that it occupied before the application of the wash water. The location of the surface of the sand during washing can be determined with the aid of a properly protected electric light on the end of a rod held in the hands of the operator,[1] by means of an inverted hollow counterweighted cone[2] connected to an indicator on the operating table or other convenient location, and by other means. The amount that the sand is expanded has been found to be more important in effecting satisfactory washing of the sand than any other factor. Too great an expansion serves only to separate the sand grains so that the amount of rubbing together is reduced, thus diminishing the effectiveness of the wash Since the amount that the sand is expanded is a function of the viscosity of the wash water and its rate of rise, since the viscosity varies with the temperature, and since the temperature varies with the season and other conditions, it follows that the rate of washing at any plant should not be constant throughout the year. Adjustable rate-of-flow controllers are sometimes installed on wash-water pipes, connected with a visible indicating gage, to enable the operator to know at what rate the filter is being washed.

572. Amount of Washing. Only experience will tell when the filter has been washed sufficiently. The appearance of the rising water in the filter and the ability of the operator to see through the water to the sand layer are usually guides on which the judgment is based. When the wash water is flowing through the filter, the sand should appear to rise evenly in all parts of the filter, and care must be taken not to wash so vigorously as to carry sand into the wash-water trough. When to stop the wash is determined principally by experience. The wash is usually stopped when the water is sufficiently clear for the sand to be visible. Upon completion of the wash, the sand should settle back into place so as to present a smooth, level surface. The duration of a wash is usually in the neighborhood of 3 to 6 min. It is desirable to use as little water for washing as possible, because of the cost of the wash water. In a properly operated plant the quantity of wash water used should not exceed about 2 per cent of the total amount of water filtered. Lower amounts have been attained, and higher amounts are frequently encountered. In general, it can be considered that satisfactory results are being obtained when the quantity of wash water does not exceed 4 per cent of the filtered water. When the filter is put back into service after washing, the loss of head through it should be in the neighborhood of 4 to 6 in.

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[1] See also GILKISON, G. F., "Controlling Wash Water Rates by Sand Expansion," *Proc. 9th Ann. Water Works School, Univ. Kansas*, 1931, p 106: and HOPKINS, E. S., "Control of Washing," *Eng. News-Record*, May 23, 1929, p. 844.

[2] See also "Eliminating the Guess from Sand Expansion Determination," *Water Works Eng.*, Jan. 5, 1938, p. 33.

573. Wash-water Pressure. The pressure at which the wash water should be applied is about 15 p.s.i. as measured at the strainer system. In the determination of the discharge pressure from the wash-water pump, or the elevation of the wash-water tank, allowance should be made for the loss of head in the pipes leading to the filters. This head loss, as well as the loss in the filter during washing, has been tested by Dixon[1] and has been shown to vary between about 10 and 63 ft., dependent upon the rate of washing.

Wash-water pressure can be obtained in any one of three ways: (1) by means of an elevated storage tank into which water is pumped continuously at such a rate as to fill the tank between washings, (2) by means of a wash-water pump that operates only when a filter is being washed, or (3) by taking water from the waterworks distribution system in the streets through a pressure-reducing valve. Both the first and the second methods are frequently used. The disadvantages of the third method are so great as to preclude its use in well-designed plants. The advantages of the first method are that only a relatively small pump is needed and that it can be operated continuously at the highest efficiency. A disadvantage is that a large, and usually unsightly, wash-water storage tank is needed. The advantage of the second method is that the tank is not required. The wash-water pump is large and expensive, however, and because of its intermittent use cannot be operated to the best economy.

574. Capacity of Wash-water Equipment. Normally the rate at which wash water is applied, where no other agitation is provided, is 2 cu. ft. of water per minute per square foot of filter surface, equivalent to a rise in the filter box of 24 in. per min. The tendency in design is toward higher rates of washing, primarily because of the larger sizes of sand being used, which require a faster application of water for equal expansion. The capacity of the wash-water storage tank should be sufficient to give a normal wash to two filters for a period of 5 to 6 min. at the maximum rate without requiring the refilling of the tank.

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[1] Dixon, G. G., "The Hydraulics of Rapid Sand Filters," *Water Works & Sewerage*, April, 1933, p. 104.

Capacities of tanks vary between 30,000 and 170,000 gal. per million gallons of filter-unit capacity. With an average capacity of about 70,000 gal.[1] per million gallons per day per filter unit.

575. Wash-air Equipment. Wash air is usually applied by means of a blower or compressor, but it may be supplied through a wash-air storage tank in which air is accumulated between washings. Wash air should be applied at a rate of about 2 to 3 cu. ft. of free air per minute per square foot of filter surface for a period of about 5 min. The air pressure is slightly greater than the depth of water above the air distribution system plus the friction in the air pipes, which is usually quite small. The wash-air storage tank should hold at least double the amount of air necessary to wash one filter. The compressor supplying the wash-air storage tank should have sufficient capacity to refill the tank between washings.

All wash-water and wash-air pumping equipment should be installed in duplicate as a factor of safety in the event of sudden breakdown and in order that repairs may be made on one pump or blower while the load is being carried by the other machine.

576. Operating Tables and Equipment. An operating table holds the equipment for the control of a filter. The valves controlled from the operating table include the following:

The influent valve.
The effluent valve
The waste-water, or sewer, valve
The wash-water valve.
The wash-air valve.
The rewash valve.

Other equipment placed on the operating table includes

A loss-of-head gage.
A sampling device.
A rate-of-flow gage

The valves may be opened or closed either manually or hydraulically or by electric motor. A hydraulically operated valve is illustrated in Fig. 164. Where the valves are operated by hand, they are controlled by handwheels placed on stands and not on the operating table. Where the valves are controlled hydraulically or electrically, the control handles or buttons are grouped on an operating table in such a manner that the operator can observe what is happening in the filter. An operating table is illustrated in Fig. 249.

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[1] "Water Treatment Plant Design," *Manual of Practice* 19, American Society Civil Engineering, 1941.

In conveying pressure from the pipes in the gallery to the gages on the table, or in protecting cables attached to hydraulic valves, copper tubing is desirable because of the rapid deterioration of steel piping.

An interesting innovation to supplant the conventional operating table has been installed in the new filtration plant at Springfield, Ill.[1] All the equipment is controlled electrically, and all the operating features of the plant are centered at one point.

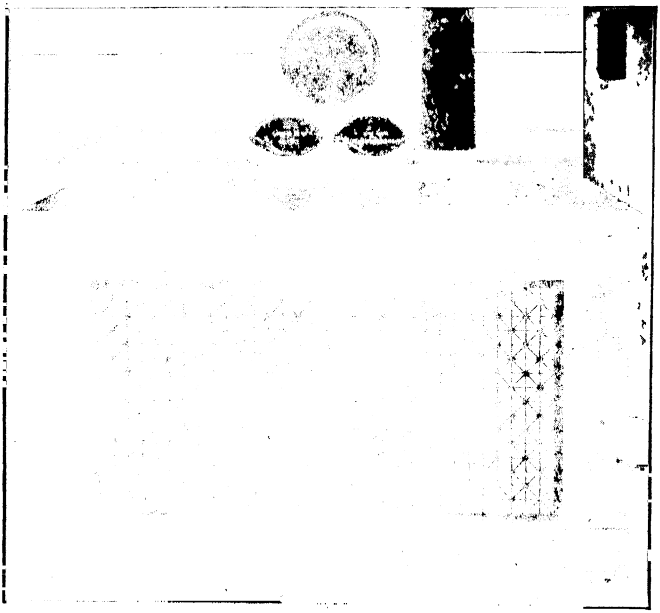

FIG. 249. Operating table for filter plant at Washington, D. C. (*Courtesy of American Water Softener Co.*)

577. Loss-of-head Gages. The simplest form of loss-of-head gage is illustrated in Fig. 247. This type of gage is not suited for general use, because the levels of the surfaces of the water in the two piezometers are usually too far apart when it is time to wash the filter to be seen from one position. In order to overcome this difficulty, it is necessary to transmit information of these two water levels to the same place so that they can be compared. Since only the difference in elevation of these two water surfaces is important, this is the information that is usually conveyed to the operator by the loss-of-head gage on the operating table. An installation of a loss-of-head gage and a rate controller are illustrated in Fig. 250.

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[1] See SPALDING, C. H., "The New Water Purification Plant at Springfield, Ill.," *Jour. A.W.W.A.*, March, 1935, p. 327.

578. Sampling Devices. As the water leaving a filter is under atmospheric pressure, it will not rise to the height of the operating table to permit sampling at a convenient point. A common method of raising the water to the table for sampling is by means of small hand pump placed on the operating table. In taking a sample, considerable water should be pumped to waste, through the drain provided on the operating table, in order to assure the collection of a representative sample.

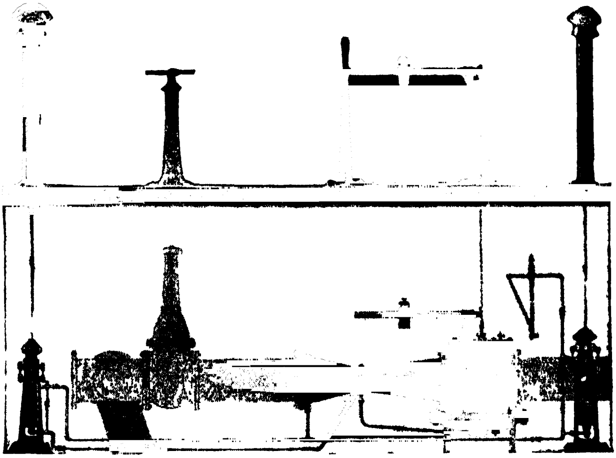

FIG. 250.—Connections for rate controller and loss-of-head gage. (*Courtesy of Simplex Valve and Meter Co.*)

579. Clear-water Basin. The clear-water basin acts as an equalizing reservoir which permits the filter to act at a constant rate. Its capacity is based on the same principles as those outlined on page 328 for an equalizing reservoir. The prediction of the fluctuations of demand to be met after the completion of a filter plant do not always justify so detailed a study as is required in the determination of the capacity of an equalizing reservoir. It is commoner practice to allow a certain number of hours of storage, usually in the neighborhood of 8 to 12 hr. Supplementary storage on the distribution system is frequently desirable.

The clear-water basin should be carefully protected against contamination. A lightproof and insectproof roof should be provided to prevent the growth of algae in the basin and to prevent access to the basin by insects which cause trouble by laying eggs in the water. Such a roof will serve also as a protection from birds and other surface sources of contamination. Manholes in the basin roof should be made watertight, but ventilation is necessary in order to avoid abnormal air pressures when the reservoir is being filled or emptied. No sewers should be permitted within the neighborhood of the clear-water basin, and the basin should be secure against leakage either from within or from without.

580. Operation of a Rapid Sand Filter Plant. The operation of a rapid sand filter plant involves considerations other than the filter alone, as the water must be prepared for filtration and its quality improved and protected thereafter. An example of thorough plant operation is given by the plant at Richmond, Va.,[1] where the following steps occur: (1) prechlorination of raw water, (2) pH adjustment in coagulation control, with lime or acid, (3) coagulation by quick mix followed by mechanical filtration, (4) sedimentation, (5) powdered activated carbon applied to the filter influent, (6) pH adjustment to 8.4 with lime, (7) residual chlorine of not less than 0.2 p.p.m. maintained at all points in the distribution system.

Routine observations that should be recorded in the control of the operation of a filtration plant should include (1) the quantity of water treated daily; (2) the length of run of each filter between washings; (3) the percentage of wash water used for each filter; (4) the quantity of chemicals used and rates of application for all purposes; (5) the alkalinity, turbidity, color, temperature, and possibly the solids in the raw and in the filtered water; (6) the number of bacteria in the raw, settled, filtered, and chlorinated water; and (7) presumptive tests for *E. coli* in the raw, settled, filtered, and chlorinated water.[2]

Conditions that must be guarded against in the operation of a rapid sand filter include (1) the formation of mud balls, (2) cracking or shrinkage of the filter bed, (3) overloading, (4) shortening of filter runs, (5) air binding, (6) loss of sand, (7) growth of size of sand particles, (8) displacement of the gravel, etc.[3]

581. Mud Balls. Mud balls consist of grains of sand and material, carried over from the coagulating basin, stuck together in clumps varying from the size of a pea up to 1 to 2 in. or more in diameter. Because of their lighter specific gravity, balls are found most densely collected at or near the top of the filter; but when of a specific gravity equal to or heavier than that of the sand,[4] they may be distributed throughout the sand and in some cases attached firmly to the gravel. Mud balls accumulate over spots where the rising swirls and velocities of the wash water are least violent, thus inducing greater unevenness in the rise of wash water and aggravating the inequality of the filter wash.[1]

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[1] ENSLOW, L. H., "Modern Water Chlorination Practice," *Jour. A.W.W.A.*, 1934, p. 579.

[2] HOOVER, C. P., "Laboratory Control for Water Works," *Jour. A.W.W.A.*, 1933, p. 1279.

[3] HUDSON, H. E., JR., "Filter Washing Experiments," *Jour. A.W.W.A.*, Vol. 27, p. 1547, 1935. BAYLIS, J. R., "Filter Bed Troubles and Their Elimination," *Jour. New Engl. Water Works Assoc.*, March, 1937, p. 1.

[4] BAYLIS, J. R., "Filter Bed Troubles and Their Elimination," *Jour. New Engl. Water Works Assoc.*, March, 1937, p. 1.

Large caked lumps of sand are more commonly found attached to the gravel or to the side walls of the filter. It is believed that the cause of mud-ball formation is insufficient washing of the sand which permits gelatinous material to adhere to the surface of the sand grains. The effect is noticed particularly in fine sands, less than 0.4 mm. in diameter, and is aggravated by the presence of microorganisms, manganese, and other substances in the water. Although a 50 per cent expansion of the sand in washing is effective in minimizing the production of mud balls, it is not a certain preventive. Control of the quality of the water applied to the filter has been found an effective preventive of mud-ball formation in some plants; *e.g.*, at South Pittsburgh mud-ball formation was eliminated when the pH of the water in the settling basin was carried at 6.9.[2]

Methods of removing mud balls and hard places after they have formed include (1) dipping them off the surface with strainers while wash water is run through the filter at a slow rate; (2) breaking them up with rakes and sharp hoes and subsequently washing off the particles; (3) washing the sand in place to break up the mud balls either with a high-velocity jet issuing from the end of a pipe with which the sand may be prodded or stirred, or by means of a high-velocity surface wash; (4) washing the filter or allowing it to stand full for some time, up to 18 hr., with a solution of some chemical, as described on page 498; and finally, (5) digging out the hard places with shovels or removing, washing, and replacing the sand.

Where the volume of the mud balls in the top 6 in. of the filter is less than 0.1 per cent of the volume of the sand in the same space, the condition of the filter may be considered excellent; but where it is 5 per cent or over, the filter is in a very bad condition.[3] Among the principal objections to the presence of mud balls in a filter is the danger from cracking and clogging of the filter bed.

582. Cracking and Clogging of Filter Bed. Cracks in the filter bed and the pulling away of the sand from the side walls are caused by the compression of the soft, gelatinous coating on the sand grains which allows the sand grains to push together as the head loss is increased during the operation of the filter. The condition is remedied by much the same methods as the formation of mud balls is overcome. The effect of such shrinkage of the filter bed is to permit dirty matter to penetrate deeply into the bed, even into the gravel, and to impair both the washing of the filter and the efficiency of filtration.

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[1] See also HOPKINS, E. S., "The Cause, Prevention, and Elimination of Mud Ball Formation," *Jour. Penna. Water Works Operators' Assoc.*, 1932, p. 74.

[2] See also BAYLIS, J. R., "A Study of Filtering Materials for Rapid Sand Filters," *Water Works & Sewerage*, January, 1935, p. 20.

[3] See also BAYLIS, J. R., "Experiences in Filtration," *Jour. A.W.W.A.*, July, 1937, pp. 1010 and 1022.

Uneven clogging of the filter usually follows the formation of mud balls, cracking of the bed, and the pulling away of the sand from the side walls. Clumps of mud balls settle into these openings, aggravating the condition and resulting in the formation of clogged areas. Effects of clogging include uneven loading, shortened runs, and inadequate purification.

583. Cleaning Rapid Sand Filters. Experience has shown that it may be necessary at times to clean the filter sand or the gravel to overcome difficulties from mud balls, cracking, and other causes. Usually cleaning serves only as a temporary expedient. The cause of clogging should be corrected. Various chemical treatments have been used, such as caustic soda, sulphuric acid, hydrochloric acid, soda ash, sulphur dioxide, and chlorine.[1] Chlorine may be used where the material to be removed is of a biological origin; copper sulphate may be effective against algae; caustic soda, for alum deposits and organic material; hydrochloric acid or carbonic acid, against calcium carbonate; and sulphur dioxide, for dissolving iron, manganese, or calcium carbonate. For small filters, H.T.H., or Perchloron, or Clorox may be used. Liquid chlorine may be used more economically on large filters. Chlorine may be applied as a solution at a concentration of about 50 p.p.m. of available chlorine. The solution should stand in contact with the sand for 4 to 24 hr., or longer. Caustic soda can be applied as a solution or, preferably, by spreading 1.0 to 3.0 lb. of the flaked material per square foot of the filter surface. The soda is dissolved by allowing the wash water to rise 4 to 5 in. above the sand.

Sulphur dioxide has been used, as a patented process (U.S. patent 2069621), to clean filter sand coated with iron or manganese. An aqueous solution of sulphur dioxide is pumped through the filter for many hours. The method is not recommended where the sand grains are coated with calcium carbonate because of the possibility of cementing the grains together with calcium sulphate or sulphite which might be formed.[2]

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[1] Geyer, J. C., "A Study of Chemicals Used for Cleaning Rapid Sand Filters," *Jour. A.W.W.A.*, May, 1937, p. 668; "Sulphur Dioxide Is Used to Clean Filter Sand and Gravel in Place," *Water Works Eng.*, Feb. 2, 1938, p. 149; and Baylis, J. R., "Filter Bed Troubles and Their Elimination," *Jour. New Engl. Water Works Assoc.*, Vol. 51, p. 1, 1937.

[2] See also Angle, C. F., "Cleaning Filter Sand with Sulphur Dioxide," *Water & Sewage Works*, March, 1947, p. 92.

Before acids or other chemicals are applied to a filter, samples of the sand to be cleaned should be tested in the laboratory with the chemical to be used. Some concentrations and periods of application of chemicals that have been used with varying degrees of success are indicated in Table 86. After chemicals have been used, the filter should be thoroughly washed to remove the loosened substances and to remove the chemicals.

TABLE 86. CHEMICALS USED FOR CLEANING FILTERS

Chemical	Amount of chemical used or strength of solution	Period of contact, hr.
Caustic soda and soda ash. .	$NaOH$ 1.8 lb. per sq. ft. Na_2CO_3 5.2 lb. per sq. ft.	18[2]
Caustic soda .	0.8-1.2 lb. per sq. ft.	22-72
Soda ash	0.2 lb. per gal.	24-36
Sulphuric acid	0.092 lb. per sq. ft.	4
Chlorine	10-12 lb. per million gallon and 50 p.p.m.	18
Sulphur dioxide[1]	1-2 per cent	16

[1] *Water Works Eng.*, Feb. 2, 1938, p. 149.

[2] Tank heated with steam.

The handling of chemicals used in the cleaning of rapid sand filters is hazardous. A special committee of the American Water Works Association has reported on precautions and methods in the handling of such chemicals.[1]

Mechanical methods of cleaning filters in place include raking, scraping the surface, use of a high-pressure water jet, ejecting the sand, and the use of steam. In some cases the sand may be removed either by hand tools or by a hydraulic ejector and cleaned outside the filter. All these methods have been found helpful in overcoming unexpected difficulties, but they should not be depended on as routine procedure. The cause of the difficulty should be discovered and remedied.

584. Maintaining Filter Runs. The length of the filter run is affected by the character of the influent water, the rate of filtration, the loss of head permitted between washings, and the thoroughness of each wash. The character of the water that is applied to the filter is under the control of the operator in so far as he is able to affect it in the coagulating basin through the application of chemicals, more satisfactory conditioning, or a change in the period of subsidence. Only experience with the particular water being treated can serve as a guide in any particular plant but, in general, an increase in turbidity or of certain microorganisms will shorten filter runs.[1]

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[1] See "Third Report of Committee on Chemical Hazards in Water Works Plants," *Jour. A.W.W.A.*, March, 1939, p. 489.

The runs may be lengthened by a more thorough pretreatment of the water or by diminishing the rate of filtration. Where too high a loss of head is permitted between washings, the length of run may be diminished by air binding resulting from the accumulation of gases thrown out of solution as a result of the reduced pressure in the filter. A remedy is the reduction of the loss of head between washings. In some plants the length of run may be increased by "bumping" the filter occasionally between washings. A filter is bumped by preparing it, in the ordinary manner, for washing and turning on the wash water for a moment to break up the *schmutzdecke.*

585. Loss of Sand. Sand is carried out of the filter by wash water usually as a result of the increase in the size of the sand grains by material that adheres to them, reducing the specific gravity of the clump and exposing more surface to the rising wash water. The loss of sand is particularly noticeable as the temperature of the wash water decreases because of the increased lifting power of the water with increasing viscosity. Since the cause of the difficulty is dirty sand, to decrease the rate of wash may serve ultimately either to aggravate the condition or to cause other troubles in the filter operation. The ultimate effective remedy is to clean the sand.

586. Growth of Size of Sand Particles. Increase in the size of sand particles in a filter is due to the quality of the raw water resulting in the deposition of a scale or of a gelatinous layer on the surface of the sand. A hard scale of calcium carbonate may form in a lime-soda softening plant as a result of incomplete recarbonation, dissolved calcium carbonate being thrown out of solution through the absence of sufficient dissolved carbon dioxide. The increase of carbon dioxide may also serve to dissolve manganese that may have deposited on the sand grains.[2] The remedy of the condition lies in the control of the quality of the water being placed on the filter.

587. Displacement of Gravel. Displacement of the gravel in a filter may result from a break in the underdrainage system, from uneven distribution of the wash water, or from the use of too high a rate of wash. The condition aggravates the uneveness of the wash-water distribution, mixes the gravel with the sand, and may permit the escape of sand into the underdrains.

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[1] See also MARSHALL, L. A., "How Does Microscopic Plant Life Affect Filter Runs," *Water Works Eng.*, Sept. 24, 1930, p. 443

[2] CARPENTER, L. V. ' Effects of Manganese on Filter Operation," *Am. Jour. Pub. Health*, October, 1931, p. 1153.

The condition may be remedied by diminishing the rate of filtration or by repairing or renovating the underdrainage system. The escape of sand into the underdrains is dangerous, because its remedy is expensive, usually involving the shutdown of the filter and the tearing up and replacing of the underdrain pipes. Gravel that has become misplaced in a filter may, sometimes, be replaced without shutting down the filter by drawing a straightedge across the surface of the gravel while wash water is running through the filter. The fine gravel, which is almost suspended in the rising wash water, is easily shifted about in this manner.

588. Results of Rapid Sand Filtration. A rapid sand filter will remove suspended matter, color, odor, and bacteria from water and will so alter the characteristics of certain polluted or otherwise undesirable waters as to make them suitable for domestic and most industrial uses. The removal of bacteria is usually insufficiently effective to place dependence on the filter alone for the production of a hygienically safe water. Where the bacterial load is at all heavy, the filtration process must be supplemented by sterilization. Raw water with turbidities as high as 35 to 40 p.p.m. can be successfully treated by rapid sand filtration. Color can be reduced to 10 p.p.m. or less.

Streeter[1] has expressed the relation between the bacterial quality of raw water and the effluent resulting from coagulation and sedimentation as $E_a = cR^n/\log T$ in which R denotes the bacterial content of the raw water, E_a that of the effluent from the settling basin, and c and n are empirical constants. When T was expressed in hours, the values of c and n for the experimental plant were found to be 0.57 and 0.88, respectively. The U.S. Public Health Service[2] has expressed the relationship between the bacterial content of influent and effluent water in purification processes as

$$y = cx^n$$

in which x and y denote, respectively, the bacterial content of the influent and of the effluent water.

Some results of the operation of rapid sand filters are shown in Table 87. A comprehensive summary of such results is given in *Public Health Bulletin* 172, 1927.

589. Permissible Loads on Filters. There is a limit to the bacterial load that a rapid sand filter can carry in safety. An investigation made by the U.S. Public Health Service disclosed that the average modern rapid sand filter plant, when aided by postchlorination of the filtered effluent, should have delivered to it a raw water having an *E. coli* index not exceeding 5,000 per 100 cc. If aided by continuous prechlorination and postchlorination, the load might be increased to 20,000 per 100 cc.

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[1] STREETER, H. W., "The Bacterial Efficiency of Certain Intermediate Stages of Water Purification," *Public Works*, December, 1933, p. 17.

[2] "The Efficiency of Water Purification Processes," *Pub. Health Bull.* 193, 1930.

In so far as the load of zinc, manganese, iron, and similar substances in the raw water is concerned, there seems to be no limit as to the amount that can be removed by purification, except the practical question of economy. No practicable method has, as yet, been devised for the removal of sodium chloride, the presence of which in the raw water supply is, therefore, limited to the amount permissible in the effluent. Other substances that should be excluded from the plant influent are the phenols and cresols and other tarry derivatives from gas and coke by-product manufacturing. The presence of decayed vegetation and of excessive growths of microorganisms is undesirable in the influent and should, therefore, be excluded.

TABLE 87.—REMOVAL OF BACTERIA BY VARIOUS WATER-PURIFICATION PROCESSES

	Cincinnati[4] *B. coli* per cubic centimeter				Louisville[4] *B. coli* per cubic centimeter				Akron,[3] per cent removal		Cincinnati,[3] per cent removal		Portsmouth,[3] per cent removal	
Raw water	685	3,150	9,190	26,400	845	3,350	7,620	30,900	Total	*B. coli*	Total	*B. coli*	Total	*B. coli*
Settled .	362	759	1,170	1,690	592	1,540	2,500	2,490	73	82	80 to 90 87.3[2]	80 to 90 83.1[1]		
Coagulated...	84	183	256	524	116	228	369	633	38	69	70 to 90 83.8[2]	70 to 80 77.5	70	
Filtered ..	10	26	43	140	9	13	16	30	81	61	80 or better	80 or better	50	
Chlorinated .	2.5	9	20	44	0.9	2.9	3.9	6	..	..	75.3[2]	84.6[3]		
All combined[1].	99.63	99.68	99.78	99.84	99.89	99.91	99.95	99.98	..	..	99.93[2]	99.95[3]		

[1] *B. coli* index, average for 1917–1921, inclusive.

[2] Average for 1917–1921, inclusive.

[3] Report, Ohio Conference on Water Purification, 1922.

[4] "Studies of the Effect of Water Purification Processes," *Pub. Health Bull.* 172, 1927.

590. Pressure Filters. A pressure filter is a rapid sand filter placed in a closed watertight tank, as shown in Fig. 251. The water passes through the sand and emerges from the filter under a pressure greater than atmospheric. The filter is operated similarly to a gravity-type filter except that the coagulated water is usually applied directly to the filter without mixing, flocculation, or conditioning. Automatic filters are available in which the valves are manipulated automatically to backwash the filter at a predetermined time or head loss.

Pressure filters are used almost exclusively on small water supplies, particularly for industrial purposes. They are particularly advantageous for installations where the water is received under pressure, as no pumping after filtration is required. Such conditions occur in buildings, industrial plants, and private estates which are supplied with water under pressure by the public water supply and in some cities, where water is conveyed to the city under pressure from the higher watershed. The use of pressure filters for a public water supply is unusual because of their high cost, the inefficiency of operation, and the relatively poor quality of the results obtained.

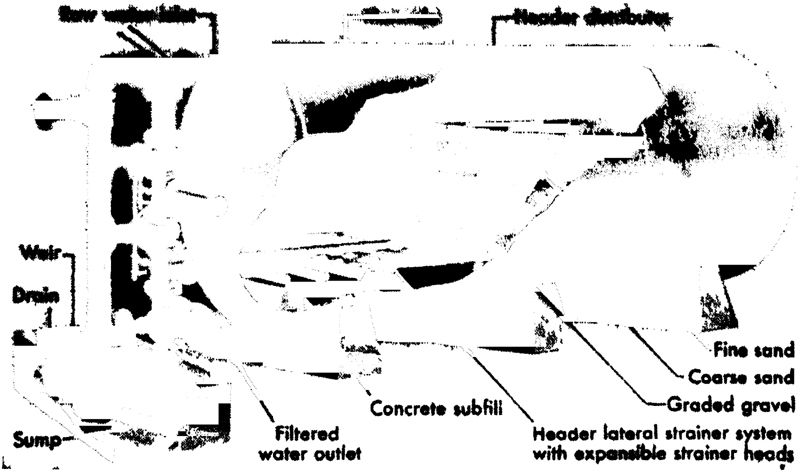

FIG. 251 Horizontal pressure filter. (*Courtesy of the Permutit Company*)

591. Double Filtration. Double filtration may consist of the filtration of water through two or more slow sand filters in series or through a rapid sand filter and then through a slow sand filter or another rapid sand filter. Where double filtration is practiced, the use of a preliminary, or "roughing," filter of the rapid sand type is common. Whatever the type of the preliminary filter, its filtering material is coarser than that in the final filters, and the rate of filtration is appreciably higher. Roughing filters of the rapid sand type seldom require the use of a coagulant, and they may be operated at rates as high as 130 gal. per min. per sq. ft., as at the Walton plant of London's water supply.

Double filtration is found useful primarily in increasing the capacity of existing slow sand filter plants where the available area of land is restricted or in the construction of slow sand filter plants on restricted areas. As a result of the preliminary treatment in the roughing filters at Walton, the slow sand filters are more than doubled in capacity.[1]

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[1] "New Walton Filters of London, England," *Water Works*, September, 1926, p 419

CHAPTER XXIX

WATER SOFTENING

592. Purposes.[1] Among the principal purposes of water softening may be included (1) conservation of soap, (2) reduction of work in the laundry, (3) prevention of the formation of scale in steam boilers, and (4) preparation of water for domestic and industrial uses. Softening increases the efficiency of filtration; aids in the removal of color, iron, and manganese; makes possible, through proper operation, the production of a noncorrosive water; improves the cooking of foods and is desirable for other domestic purposes; and increases the efficiency of reduction of bacteria when filtration follows softening. Where soap is used in hard water, an undesirable precipitate of calcium and magnesium compounds, combined or mixed with other substances, is formed which deposits a scum or scale that is difficult to remove. Water must be reduced to zero hardness before soap will become effective as a detergent. That it is more economical to soften water with chemicals than with soap was demonstrated by Hudson. Where the cost of soap is 20 cents per lb., of soda ash is 1½ cents per lb., and of lime, $Ca(OH)_2$, is ½ cent per lb., the relative costs of soap to soda ash or lime, per pound of hardness as calcium carbonate, are roughly 300:60:1.[2] The cost of domestic water softening, as compared with municipal water softening, may be reduced by by-passing hard water to be used for flushing, sprinkling, and similar purposes.

Despite its demonstrated advantages and economies the field of municipal water softening is relatively undeveloped; only about 20 per cent of the municipalities in the United States in which the hardness of the water supply is greater than 110 p.p.m. are softening their water. A census of all the municipal water-softening plants in the United States, showing pertinent data concerning their operation, is presented by H. M. Olson.[3]

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[1] HUDSON, H. W., "Soap Usage and Water Hardness," *Ill. State Water Survey*, *Circ.* 13, 1934; WALKER, W. H., "Municipal Water Softening," *Jour. A.W.W.A.*, January, 1934, p. 77. GREEN, H. R., *Jour. A.W.W.A.*, Vol. 25, p. 982, 1933; and OLSON, H. M., "Benefits and Savings from Softened Water for Municipal Supply," *Jour. A.W.W.A.*, April, 1939, p. 607, with comprehensive bibliography.

[2] "Alkalies and Chlorine in the Treatment of Municipal and Industrial Water," *Tech. Bull.* 8. Solvay Technical and Engineering Service, 1941.

[3] *Jour. A.W.W.A.*, December, 1941, p. 2153.

593. Results Desired. In the operation of a municipal water-softening plant it is now customary to reduce the carbonate hardness to 35 to 40 p.p.m. and the total hardness to about 50 to 100 p.p.m. The solubility of calcium carbonate in softened water is an important criterion of its desired quality. Water that is supersaturated with calcium carbonate will deposit a coating on pipes and sand grains, whereas water that is undersaturated is apt to be corrosive. The solubility equilibrium of calcium carbonate is controlled by the lime–soda ash dose, the addition of alum, recarbonation, or the use of sodium hexametaphosphate (Calgon). The carbonate balance of the water is determined as explained in Sec. 660. In general, it is considered good practice to carry a calcium carbonate alkalinity of about 25 p.p.m., which necessitates a pH of 8.2 to 8.5. If the pH is low, soda ash should be added, and if too high, alum or carbon dioxide should be added.

594. Methods of Softening Water. Public water supplies are softened by either the lime-soda process or the zeolite (base-exchange) process. Water to be used for industrial purposes may be softened by the above methods or by the use of synthetic organic detergents, the so-called sulphonated oil soaps and related substances, and proprietary preparations usually containing organic compounds

595. The Lime-soda Process. A flowing-through diagram of a municipal water-softening plant is shown in Fig. 252. The process involves the thorough mixing of the chemical with the water, followed by slow agitation for 30 to 60 min. to allow completion of the chemical reaction. The proper time to be allowed is greatly affected by the temperature because the chemical reactions and sedimentation take place rapidly at higher temperatures. When the temperature of the water approaches 39°F., the settling area needs to be double that at 71°F. Precipitated chemicals are removed by sedimentation or filtration, or both. The water may be recarbonated after sedimentation to restore the carbonate balance.

The process may be operated intermittently or continuously, either cold (50-70°F.) or hot (212°F.). Intermittent softening involves batch or fill-and-draw operation in wood, steel, or concrete tanks of any convenient shape, from which sludge is withdrawn either by gravity in a conical- or pyramidal-shaped bottom, or by mechanical scrapers from flat-bottom tanks.

The continuous cold process is most commonly used in municipal practice. For industrial purposes intermittent operation may be used and, where the softened water must be heated, as for boiler feed-water water conditioning, the hot process is most economical. Boiling reduces the carbonate hardness by driving off the dissolved carbon dioxide,

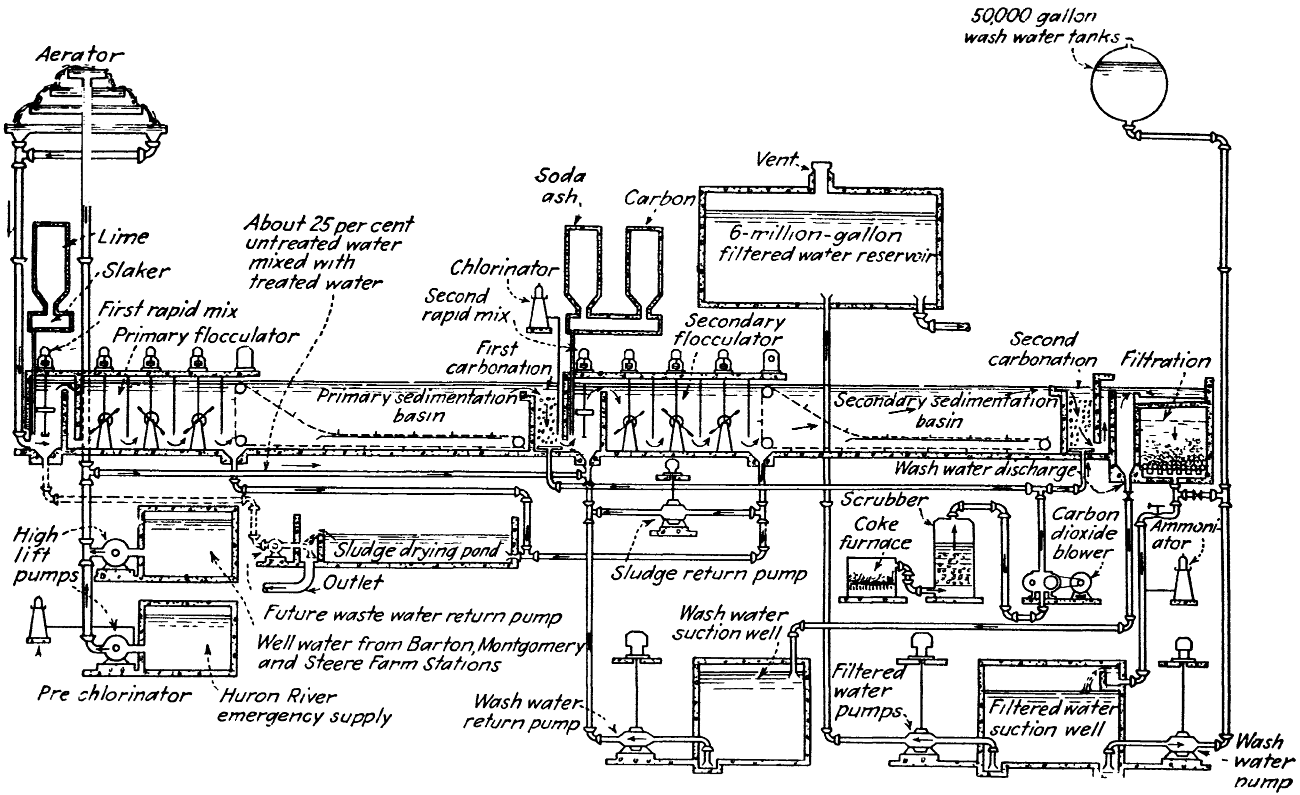

Fig. 252.—Flowing-through diagram of municipal water-softening plant at Ann Arbor, Mich. (*From H. E. McEntee, Jour. A.W.W.A., September*, 1940. *p.* 1602.)

resulting in the precipitation of insoluble carbonates. The complete softening of boiler feed water requires the use of excess chemicals and the unit cost may be higher than for municipal water softening.

596. Chemistry of the Lime-Soda Process. The following typical reactions occur when lime is added to hard water:

$$\underset{\substack{\text{Lime,}\\ \text{insoluble}}}{CaO} + \underset{\substack{\text{carbonic}\\ \text{acid}}}{2H_2CO_3} = \underset{\substack{\text{calcium bicarbonate,}\\ \text{soluble, causes hardness}}}{Ca(HCO_3)_2} + \underset{\text{water}}{H_2O} \qquad (1)$$

$$\underset{\substack{\text{Calcium bicarbonate}\\ \text{soluble, causes hardness}}}{Ca(HCO_3)_2} + \underset{\text{lime}}{Ca(OH)_2} = \underset{\substack{\text{calcium carbonate,}\\ \text{insoluble}}}{2CaCO_3} + \underset{\text{water}}{2H_2O} \qquad (2)$$

$$\underset{\substack{\text{Calcium sulphate,}\\ \text{insoluble, causes}\\ \text{hardness}}}{CaSO_4} + \underset{\substack{\text{sodium carbonate}\\ \text{or}\\ \text{soda ash}}}{Na_2CO_3} = \underset{\substack{\text{calcium}\\ \text{carbonate,}\\ \text{insoluble}}}{CaCO_3} + \underset{\substack{\text{sodium sulphate,}\\ \text{soluble, does not}\\ \text{cause hardness}}}{Na_2SO_4} \qquad (3)$$

Reaction (1) expresses the conditions that occur in nature to cause hardness. The calcium carbonate of the limestone is dissolved by water containing dissolved carbon dioxide. Reaction (2) is the reaction that occurs when temporary hardness is removed by the addition of lime. Reaction (3) occurs when permanent hardness is removed by the addition of soda ash. Normally, reaction (3) would be followed by reaction (2) in order to remove all hardness.

The amount of lime and sodium carbonate necessary to complete these reactions is dependent on the amount of calcium and magnesium dissolved in the water and on such conditions as temperature, return of precipitated sludge, thoroughness of mixing, and time of contact. The necessary amount of chemical can be computed approximately by stoichiometrical computations.[1] It is not possible to determine, by such computations, the precise amount of chemical required because of other substances present in the water. Various rational and empirical formulas, all based on stoichiometrical principles, have been devised to express the quantities of lime and soda ash needed.[2] Larson and Buswell[3] have shown that a residual hardness of about 20 to 27.4 p.p.m. is the theoretical minimum to which hardness can be reduced by the cold process using only lime and soda ash. Softening under higher temperatures is more complete.

597. Chemicals Used. Either hydrated lime, $Ca(OH)_2$, or quicklime, CaO, may be used in water softening, the choice depending on the available equipment for handling the chemicals and their relative cost. Commercial grades of hydrated lime and of quicklime are about 90 per cent pure, with calcium carbonate and silica as the principal impurities. Hydrated lime keeps better in storage and it does not require the slaking equipment needed for quicklime. The latter, however, is less bulky and costs less per unit weight of calcium oxide and is, therefore, more economical for larger plants.

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[1] HOOVER, C. P., "Water Supply and Treatment," *National Lime Assoc., Bull.* 211, 1934.

[2] "Alkalies and Chlorine in the Treatment of Municipal and Industrial Water," *Tech. Bull.* 8, Solvay Technical and Engineering Service, 1941.

[3] LARSON, C. C., and A. M. BUSWELL, "Theoretical Limits of the Lime-soda Method of Water Softening," *Ind. Eng. Chem.*, January, 1940, p. 130.

In the preparation of the chemical to be added to the water it is better to prepare a well-agitated milk-of-lime suspension rather than a solution of lime water because of the relatively large volume of lime water required to give adequate dosage of lime. A difficulty involved in the use of milk of lime, however, is the continuous agitation required to prevent stratification and sedimentation in the storage tank and in pipes. A trough with an easily removable top or a soft rubber hose that is accessible for pressing or kneading in order to break down scale or to move deposits is preferable to inflexible pipe for conveying milk of lime. If high temperatures occur in slaking quicklime, the slaker should be provided with a ventilating hood in order to remove vapors.

The slaking of quicklime to produce a satisfactory milk-of-lime solution requires about 3 to 5 lb. of water per pound of lime. In practice the quicklime, after being crushed to a size convenient to be handled by the available equipment, is applied through a dry feeder to a covered container in which the lime is slaked at a temperature of about 200°F., requiring about 15 to 20 min. for complete reaction during which the milk of lime is thoroughly stirred.

It apparently makes no difference whether the lime and soda ash are mixed together or fed as one solution. The measurement and feeding of the chemicals may be controlled either volumetrically or gravimetrically by dry-feed machines which control the dose with precision. In practice, charges of lime and of soda ash are made up in batches sufficient to last for one 8-hr. period, or watch. Solutions should not contain more than about 5 per cent of the active ingredient, because stronger concentrations decrease the accuracy of feeding, although 10 per cent milk-of-lime solutions are used in practice.

Commercial grades of soda ash contain about 99.5 per cent of Na_2CO_3, and are relatively easily handled when dry. Soda ash is relatively soluble in water, forming a clear, stable solution that requires no agitation to prevent settling or stratification. Caustic soda, NaOH, reacts in a manner similar to lime but liberates an equivalent quantity of sodium carbonate. It thus replaces both lime and soda ash. With highly alkaline waters it may serve as a complete softening agent.

598. Sedimentation Basins. Because of the relatively large volume of sludge accumulated in the softening process, sedimentation basins with mechanical devices for continuous removal of the sludge are ordinarily used. One form of such basins, commonly called "clarifiers," is illustrated in Fig. 253. In general, the period of retention lies between 2 and 4 hr.

Proprietary devices, such as the Spaulding precipitator[1] illustrated in Fig. 228, combine the functions of mixing and sedimentation. The device consists of a cylindrical tank in which a central truncated cone with its small end up is inside another truncated cone with its small end down. Mixing is done in the inner cone, which is provided with a mechanical stirring device and which has a detention period of about 30 min. After mixing, the water passes through a V-shaped annular slot between the inner and the outer cone and rises with a velocity of 4 to 8 ft. per hr.[2] through a blanket of sludge that acts as an adsorbent medium and effects removal of precipitated solids. Sludge is drawn off continuously and slowly enough to maintain the essential blanket of sludge.

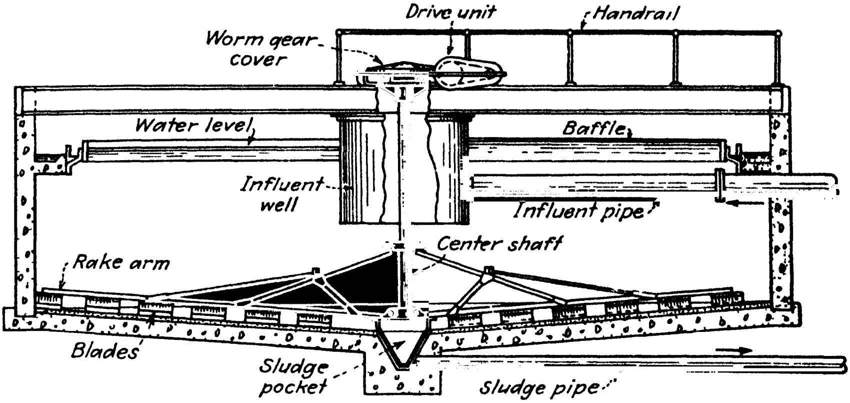

FIG. 253. Sectional elevation through a Type A circular Dorr clarifier. (*Courtesy of the Dorr Co.*)

599. Recarbonation. The addition of lime to water reduces the carbonate saturation index and increases the tendency of the water to deposit calcium carbonate on filter sand, pipes, boiler tubes, etc. The carbonate balance may be partly or completely restored and other advantages may be gained by the recarbonation of the water.[3] This is usually accomplished by diffusing carbon dioxide gas through the water or by under-water combustion.[1]

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[1] See SPAULDING, C. H., "Conditioning of Water Softening Precipitates," *Jour. A.W.W.A.*, November, 1937, p. 1697.

[2] "Alkalies and Chlorine in the Treatment of Municipal and Industrial Water," *Tech. Bull.* 8, Solvay Technical and Engineering Service, 1941.

[3] See also SPAULDING, C. H., "Water Softening Practice Shows Rapid Growth," *Eng News-Record*, June 8, 1933, p. 747; HOOVER, C. P., "Recarbonation of Softened Water," *Ind. Eng. Chem.*, Vol. 19, p. 784, 1927.

The gas is usually applied after the water has passed through the settling basins. The reaction time allowed in the recarbonation basin should be at least 20 min. Unfortunately recarbonation does not always return the water to chemical balance. For example, water with a solids content of 300 p.p.m. and calcium of 20 p.p.m. to be in chemical balance should have a pH of 8.9. Recarbonation will reduce the pH to 8.3, requiring an alkalinity, for chemical balance, of 130 p.p.m.—almost four times that required at a pH of 8.9. To avoid redissolving of precipitated carbonates, recarbonation should not be carried below a pH of 9.2 to 9.7. Although such water may be incrusting it is preferable to water that is corrosively aggressive.

The recarbonation of water containing excess lime will form a colloidal precipitate of calcium carbonate that is less soluble at a pH of 9.4 to 9.6. It is desirable, therefore, not to carry the recarbonation below this point if minimum hardness is desired. Equipment for the production of carbon dioxide consists of a burner, a scrubber, a compressor, and necessary appurtenances. The maximum amount of carbon dioxide that might be required should not exceed 500 lb. per million gallons of water treated.

A period of settling should be provided before and after recarbonation to remove the sludges formed by softening and to remove any possible precipitate formed by the neutralization of excess caustic alkalinity [$Ca(OH)_2$] by the carbon dioxide. The secondary period of sedimentation is, however, commonly omitted.

600. Disposal of Sludge. The large quantity of sludge accumulated in the lime-soda process of water softening presents a problem in its disposal. Many expedients have been adopted for its disposal, such as (1) the discharge into a stream providing sufficient dilution and carrying capacity to prevent the formation of sludge banks or the discoloration of the stream; (2) temporary storage to permit discharge into the stream during high-water period; (3) the filling of depressions or excavations, as old quarries; (4) the discharge into the city sewers through the sewage-treatment plant; (5) drying the sludge and using it for liming soil; (6) reburning for the production of lime; and (7) use as a filler for paints.

601. Operating Expedients. Greater economy and better results can be obtained by various expedients during operation that combine or modify the normal steps in the water-softening process. These expedients include (1) the use of excess lime, (2) the use of aluminum or ferrous compounds, (3) the recirculation of sludge, (4) the softening of only a part of the water, and (5) the combination of the lime-soda process and the zeolite process.

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[1] See also SCOTT, L. H., and F. C. ABBOTT, "Submerged Combustion," *Water Works & Sewerage*, October, 1939; and SCOTT, L. H., "Development of Submerged Combustion for Recarbonation," *Jour. A.W.W.A.*, January, 1940, p. 93.

The addition of an excess of 2 to 3 grains of lime per gallon followed by the addition of sufficient soda ash to react with the noncarbonate hardness and the excess lime has been found to precipitate magnesium almost completely. The presence of an excess of lime can be determined by the test for alkalinity. If the phenolphthalein alkalinity is equal to one-half of the methyl orange alkalinity, only normal carbonate alkalinity is present. If the phenolphthalein alkalinity is greater than one-half of the methyl orange alkalinity, the water contains caustic alkalinity or excess lime.

Aluminum compounds are sometimes added to the water to coagulate finely divided precipitates, thus shortening the reaction and settling periods that might otherwise be required. Compounds used include alum and sodium aluminate; a dosage of 0.5 to 1.0 grain of alum per gallon or 0.5 grain of sodium aluminate per gallon may be used Ferrous sulphate may be used in place of alum. The use of alum slightly increases the sulphate hardness, necessitating the use of additional soda ash. The reduction of hardness by sodium aluminate may be greater than would be calculated, possibly due to excess hydroxide ions released by the hydrolysis of sodium aluminate to aluminum hydroxide.

The recirculation of water-softening sludge alone, or combined with some finely divided insoluble compound, such as bleaching clay, applied to the water with the milk of lime, has been found beneficial in many plants.[1] The amount of returned sludge should be sufficient to maintain about 10,000 p.p.m. of suspended solids in the mixing chamber, but the exact number for any particular plant must be determined by experience. The procedure has been found particularly beneficial when the bicarbonate has been changed to normal carbonate, the magnesium has been precipitated as magnesium hydroxide, and the excess caustic lime, $Ca(OH)_2$, has been neutralized to carbonate. The recirculation of sludge may be combined with the excess-lime treatment.

Recirculation of sludge may result in an effluent containing 50 to 70 p.p.m. of hardness, approximately 30 p.p.m. less than would be obtained without recirculation. The Spiroacter, a proprietary device,[2] produces an effluent of 50 to 70 p.p.m. of hardness by the cold-lime process with a period of retention between 6 and 12 min., compared with 4 hr. in the conventional cold-lime softening process without recirculation of sludge and 1 hr. with recirculation.

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[1] See also HOOVER, C P, "A Recent Development in Lime Water Softening," *Rept. 8th Ann., Water Works School, Univ Kansas*, 1930.

[2] "Water Conditioning," The Permutit Co , 1943.

In some cases it has been found desirable, and economical in the use of chemicals, to reduce the hardness of a portion of the water to a degree below that required and to mix the softened water with a portion of unsoftened raw water to produce the desired degree of hardness. This method is known as "split treatment."

A combination of the lime-soda process with the zeolite process may be found economical, particularly where the sulphate hardness is high The carbonate hardness is first removed by lime treatment. The water is then recarbonated, to prevent incrustation of the zeolite sand, and the water is passed through the zeolite softener to remove sulphate hardness. Since the zeolite also removes residual carbonate hardness, the hardness of the effluent is zero.

602. A Softening Plant. Municipal softening plants are usually most economically designed to meet local conditions. Small plants and equipment for many industrial purposes may be selected from standard designs offered by equipment manufacturers. Olson,[1] in 1945, reported costs of softening plants under 1 m.g.d. capacity to be $75,000 per million gallons per day; and from $50,000 to $55,000 per million gallons per day for plants of 3 to 4 m.g.d. capacity; and from $25,000 up for plants from 5 to 50 m.g.d. capacity.

A typical small softening plant is illustrated in Fig. 252.[2] Such a plant, with a capacity of 250,000 gal. per day, would consist of (1) a mixing basin 4 by 25 ft. in plan and 12 ft. deep, the mixing being done by paddles revolving on a horizontal shaft at right angles to the direction of flow of the water, (2) a settling basin 25 by 25 by 12 ft., equipped for the continuous removal of sludge; (3) a recarbonation basin 2 by 25 by 8 ft., equipped with perforated pipes for the diffusion of carbon dioxide; (4) two gravity sand filters, each with an area of 50 sq. ft.; and (5) a secondary recarbonation tank about 5 ft. square and 5 ft. deep, which will act also as a clear well.

In small softening plants, precise chemical control is important because of the high percentage of error in the treatment resulting from small errors in the rate of application of chemicals. Precise control may be obtained by automatic equipment, which will apply the chemicals in proportion to the quantity of water being treated.

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[1] See also OLSON, H. M., "Development and Practice of Municipal Water Softening," *Jour. A.W.W.A.*, October, 1945, p 1002

[2] See HOOVER, C. P., "Small Softening Plants," *Water Works Eng.*, May 26, 1937, p. 734.

SOFTENING WITH ZEOLITES

603. Characteristics of Zeolites. A zeolite is a chemical compound so imperfectly bound together that its composition will change depending upon the concentration of chemicals in solution in its presence. Those zeolites most commonly used in water softening are a natural substance known as greensand and various forms of synthetic sodium zeolites. Greensand deposits are widely distributed with almost inexhaustible deposits in New Jersey. The zeolites used in water softening are so composed that the sodium radical of the zeolite is interchangeable in either direction for calcium and magnesium. Some of the characteristics of various available zeolites are listed in Table 88.

TABLE 88.—CHARACTERISTICS OF ZEOLITES

Name or type	Description	Exchange value, grains of $CaCO_3$ per cubic foot	Pounds of salt per cubic foot for regeneration
Glauconite, or greensand	Greenish-black, kidney-bean-shaped granules A natural zeolite	2,700 to 3,800	1 4
Precipitated synthetic, or wet, process	Sand	6,000 to 20,000	4 to 8
Fused synthetic	Not widely used	4,000 to 8,000	4
Clay group	Granulated clay[1]	5,000 to 8,000	4

[1] Porous and consequently affected by turbidity. Not durable and hence not widely used.

Both glauconite and a synthetic zeolite, known as Permutit, are widely used in practice. Permutit is a precipitated synthetic compound protected by patent rights. It is an artificial sodium zeolite ($NaAlSiO_4 \cdot 3H_2O$) that will exchange the sodium for calcium. It has the appearance of coarse sand with even-size, hard lustrous grains. It will absorb moisture from the atmosphere and must, therefore, be stored in a dry place.

Characteristics desirable in a zeolite include (1) high operating exchange value, which minimizes the size of the equipment required; (2) freedom from aggressive attack by the water being treated; (3) freedom from injury when overrun; (4) low salt requirements for regeneration; (5) availability and low first cost; (6) durability to resist the wear of rewashing; (7) freedom from the effects of turbidity, iron, manganese, or other substances not causing hardness in the water being treated; and (8) a grain of proper size, neither so fine as to cause too great a loss of head in operation or in backwashing, nor so coarse as noticeably to diminish its exchange value.

Exchange value and resistance to aggressive attack in precipitated synthetic zeolites are indicated by the chemical composition. The composition of such zeolites may be expressed as the ratio of Na_2O: Al_2O_3:SiO_2. Thus, a 1:1:6 zeolite is one containing 1 mol of Na_2O, 1 mol of Al_2O_3, and 6 mols of SiO_2. Most of the synthetic zeolites on the market fall within the range 1:1:4 and 1:1:15. In general, other things being equal, an increase in the SiO_2 content decreases the exchange value of the material but increases its resistance to aggressive attack.

The glauconites or greensands usually have a lower exchange value but are more rugged than the precipitated synthetic zeolites. In domestic installations, where the volume of zeolite involved is small and the intensity of service is low compared with that in industrial and municipal use, the precipitated synthetic zeolites are advantageous. Where the unsoftened water has not been treated, properly stabilized greensands may be desirable, as they are less affected by turbidity, iron, or other foreign substances; but, unfortunately, because of the relatively low exchange value, they may demand undesirably large equipment.

The usual salt consumption by greensand is 0.5 lb. of salt per 1,000 grains of hardness (as $CaCO_3$) removed. With an efficient gel zeolite, the salt consumption may be as low as 0.30 to 0.35 lb. per 1,000 grains removed. The head loss through a bed of synthetic zeolite is usually lower than through a greensand bed because of the larger and more irregular-shaped particles in the greensand.[1]

604. Hydrogen Exchangers. A hydrogen exchanger is a substance that has the power to exchange hydrogen ions for metallic ions. A proprietary substance with these properties, known as Zeokarb,[2] is nonsiliceous and is resistant to aggressive chemical attack. When regenerated with acid it exchanges hydrogen ions for metallic ions, such as Na, Ca, and Mg, and converts carbonates and bicarbonates to carbonic acid, chlorides to hydrochloric acid, and sulphates to sulphuric acid. It produces an effluent of lower solids content than when sodium chloride is used for regeneration. Undesirable acidity in the water treated by Zeokarb may be removed by (1) diluting with raw water, (2) neutralization with alkaline substance, or (3) absorbing with "De-Acidite," a proprietary zeolite-like substance regenerated with sodium carbonate.

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[1] See also BEHRMAN, A. S., "Progress in Municipal Zeolite Water Softening," *Jour. A.W.W.A.*, 1934, p. 618.

[2] See also APPLEBAUM, S. P., and R. RILEY, *Ind. Eng. Chem.*, January, 1938, p. 80.

The effluent from the last reaction produces only water and carbon dioxide, giving a water of a quality equal to the best of ordinary distilled water.

The reactions that occur are as follows:[1]

$$(\text{Ca, Mg, or Na}_2)(\text{HCO}_3)_2 + \text{H}_2\text{Z} = (\text{Ca, Mg, or Na}_2)\text{Z} + 2\text{H}_2\text{O} + 2\text{CO}_2 \tag{4}$$

$$(\text{Ca, Mg, or Na}_2)(\text{SO}_4 \text{ or Cl}_2) + \text{H}_2\text{Z} = (\text{Ca, Mg, or Na}_2)\text{Z} + \text{H}_2\text{SO}_4 \text{ or } 2\text{HCl} \tag{5}$$

$$(\text{Ca, Mg, or Na}_2)\text{Z} + \text{H}_2\text{SO}_4 = \text{H}_2\text{Z} + (\text{Ca, Mg, or Na}_2)\text{SO}_4 \tag{6}$$

$$\text{D} + (\text{H}_2\text{SO}_4 \text{ or } 2\text{HCl}) = \text{D}(\text{H}_2\text{SO}_4 + 2\text{HCl}) \tag{7}$$

$$\text{D}(\text{H}_2\text{SO}_4 \text{ or } 2\text{HCl}) + \text{Na}_2\text{CO}_3 = \text{D} + \text{Na}_2(\text{SO}_4 \text{ or Cl}_2) + \text{H}_2\text{O} + \text{CO}_2 \tag{8}$$

Where the zeolite is used for the softening of water, the approach of the zeolite to exhaustion can be indicated by the test for alkalinity. The exhausted zeolite is regenerated by acid, not by sodium chloride.

605. Cation and Anion Exchangers.[2] Cation and anion exchangers are being introduced, principally for the processing of industrial waters, not being restricted to softening alone. The former are regenerated with acid, the latter with caustic solution. Anion exchangers are especially suitable for removing alkali salts in brackish water. The cation exchangers are especially suited to the softening of waters of high alkalinity and for recarbonating lime-softened water. When operated for softening the cation exchanger may be regenerated either by sodium chloride or by an acid. Its capacity on either the sodium or hydrogen cycle is about 1,000 grains of hardness per cubic foot as calcium carbonate. For regeneration it requires 0.4 to 0.5 lb. of NaCl or 0.45 lb. of 66° Bé. acid per 1,000 grains of hardness, expressed as CaCO_3. The rating of the anion exchanger depends on the quality of the raw water and on operating conditions. It has an exchange capacity of about 30,000 grains of hardness per cubic foot. Where used for softening it may be regenerated with soda ash, requiring about 0.46 lb. per 1,000 grains of hardness.

The water to be treated by such exchangers should have a turbidity of less than 5 to 10 p.p.m.

606. Softening with Sodium Zeolites. Water containing either, or both, carbonate and sulphate hardness can be softened by passing it through a layer of sodium zeolite at a temperature not to exceed 100°F., so as not to injure the zeolite. Calcium and magnesium are exchanged for sodium in accordance with the following reactions:

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[1] Z means zeolite, D means "De-Acidite."

[2] See also GOUDEY, R. F., "Removal of Salts from Water," *Jour. A.W.W.A.*, March, 1940, p. 435.

$$\underset{\substack{\text{Sodium}\\ \text{zeolite}}}{NaZ^*} + \underset{\substack{\text{Calcium or}\\ \text{magnesium bi-}\\ \text{carbonate,}\\ \text{hardness}}}{\left.\begin{matrix}Ca\\ Mg\end{matrix}\right\}(HCO_3)_2} = \underset{\substack{\text{Calcium}\\ \text{magnesium}\\ \text{zeolite}}}{\left.\begin{matrix}Ca\\ Mg\end{matrix}\right\}Z} + \underset{\substack{\text{Sodium bicarbon-}\\ \text{ate, soluble,}\\ \text{not hardness}}}{2NaHCO_3} \qquad (9)$$

$$\underset{\substack{\text{Sodium}\\ \text{zeolite}}}{Na_2Z^*} + \underset{\substack{\text{Sulphate}\\ \text{hardness}}}{\left.\begin{matrix}Mg\\ Ca\end{matrix}\right\}(SO_4)} = \left.\begin{matrix}Ca\\ Mg\end{matrix}\right\}Z + \underset{\substack{\text{Sodium sulphate,}\\ \text{soluble,}\\ \text{not hardness}}}{Na_2SO_4} \qquad (10)$$

$$\underset{\substack{\text{Calcium}\\ \text{zeolite}}}{CaZ^*} + \underset{\substack{\text{Sodium}\\ \text{chloride}}}{NaCl} = \underset{\substack{\text{Sodium}\\ \text{zeolite}}}{NaZ} + \underset{\substack{\text{Calcium}\\ \text{chloride}}}{CaCl} \qquad (11)$$

Reactions (9) and (10) are softening reactions. Reaction (11) is regeneration. In reactions (9) and (10) the sodium passes off with the water as sodium sulphate or sodium carbonate, neither of which causes hardness. The effect is to reduce the hardness almost to *zero*. In municipal practice it is the custom to soften only a portion of the supply to zero hardness, mixing it with unsoftened water so that the resulting hardness is about 50 to 100 p.p.m., averaging about 85 p.p.m.

Upon the exhaustion of the zeolite, the flow of the hard water is cut off, and a solution of NaCl is allowed to stand in contact with the zeolite in the softener. The sodium in the brine replaces the calcium and magnesium in the exhausted zeolite, which is thus restored, and the calcium and magnesium are discharged to waste with the wash water. The zeolite is now ready for use for softening more water. The process may be carried on indefinitely withour renewing the zeolite. In practice it may be anticipated that about 5 per cent of the zeolite will disintegrate and wash away annually. The principal expense in connection with the process is the cost of salt and of wash water.

607. Limitations of Zeolite Softening Process. The zeolite process is not applicable to turbid, iron-bearing, or acid waters. Turbid waters deposit material on the surface of the zeolite so as to reduce its exchange value. Iron-bearing or manganese-bearing waters either deposit hydroxides on the surface of the zeolite or react with the zeolite to substitute iron or manganese for sodium in the zeolite, an irreversible reaction. Acid waters irreversibly substitute hydrogen for sodium in the zeolite or may aggressively attack the zeolite by dissolving alumina or silica from it, or they may reduce the chemical equilibrium to a new level of base exchange capacity.[1]

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* Z stands for zeolite.

[1] See also BEHRMAN, A. S., and H. GUSTAFSON, "Effect of Low-pH Waters on Zeolites," *Ind. Eng. Chem.*, November, 1936, p. 1279.

The capacity of zeolites in terms of calcium carbonate removed depends, to some extent, on the weight of sodium chloride used per unit volume of zeolite. The capacity rises with increased salt concentration up to a maximum limit. The capacity of zeolites to remove magnesium hardness is somewhat less than to remove calcium hardness, and it is also diminished in excessively hard waters or in waters high in sodium salts. The diminished capacity may be compensated for, in operation, by the use of more sodium chloride in regeneration.

Zeolite softeners require intelligent care in their operation to avoid injury to the zeolite, to the equipment, or to the quality of the water. Collins[1] recommends that the bed of zeolite be flushed annually with chlorinated water to destroy bacterial growths; that a regenerated softener not in operation should never be left full of water for more than 10 hr.; that when a softener is not in use for more than a week, the zeolite should be left *unregenerated*, and the unit should be completely drained; that synthetic zeolites, when not in use, should be kept damp but not submerged; that the hardness of the water used for washing should not exceed 10 grains per gal.; and that the raw water should be neither acid nor caustic in character.

608. Zeolite Water Softeners. A zeolite water softener resembles a rapid sand filter of either the gravity-filter or pressure-filter type, except that the customary layer of filter sand is replaced by 30 to 75 in., or more, of zeolite sand and there is a slight difference in the piping arrangements. Depths of zeolite sand in the neighborhood of 30 in. are most commonly used. Although the flow of water through the zeolite bed may be either upward or downward, the latter is more common in practice as there is less danger of carrying zeolite out with the effluent and the filtering action of the bed is better. Softeners of the pressure-filter type are more commonly used in industrial or domestic service than for public water supplies. The pressure-type softener is sometimes equipped with automatic regeneration controlled to regenerate either at fixed periods of time or in accordance with the volume of water treated.

In addition to the customary influent, effluent, and wash-water pipes it is necessary to provide a salt-solution inlet and a salt-solution-and-rinse-water outlet. Both inlet and outlet are connected to the underdrainage system. A section through a zeolite softener of the gravity type is shown in Fig. 254.

609. Size of Zeolite Softeners. The factors affecting the volume of zeolite required in a softener may be formulated as

$$Z = \frac{KQH}{G} \tag{12}$$

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[1] See also COLLINS, L. F., "A Study of Contemporary Zeolites," *Jour. A.W.W.A.*, October, 1937, p 1472

where Z = volume of zeolite to be used, cu. ft.
Q = volume of water to be treated between regenerations, gal.
H = hardness of water, grains per gal.
G = hardness to be removed between regenerations, per unit volume of zeolite, grains per cu. ft.
K = a factor whose value depends on the units used

Glauconite weighs about 90 lb. per cu. ft., synthetic zeolites about 50 lb. per cu. ft., and salt for regeneration about 60 lb. per cu. ft. Saturated lime solution contains about 26 per cent salt. These units give a measure for the estimation of space and weight requirements in equipment and plant design.

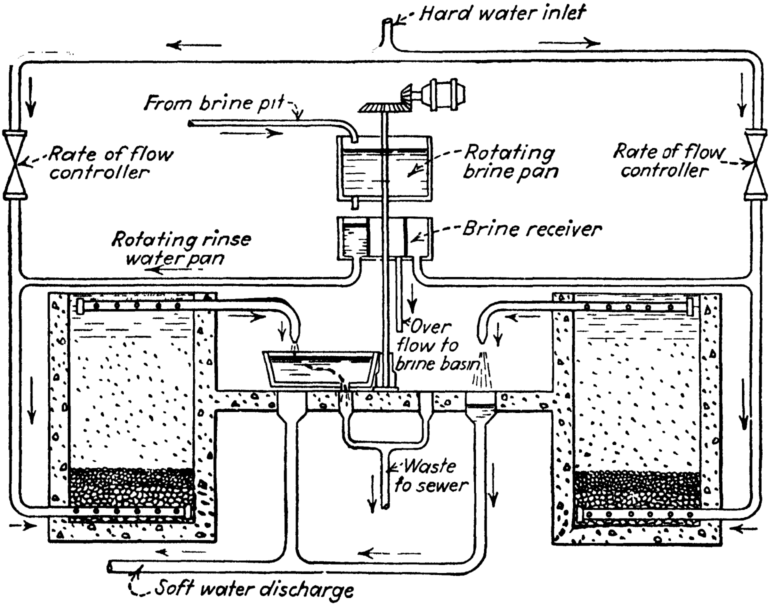

FIG. 254.—Automatic gravity zeolite water softener. (*Jour. A.W.W.A., September*, 1932, p. 1384.)

The rate of softening, regardless of the hardness of the water, is generally in the neighborhood of 2 g.p.m. per sq. ft. of zeolite filter surface. In some cases rates two to two and a half times this have been used successfully in greensand and four times as high in synthetic zeolites.[1] The thicker the bed, the greater the permissible rate of softening, but a rate of over 2 g.p.m. per sq. ft. is undesirable because of the high velocity through the bed which may prevent all the water from coming into contact with the zeolite grains for a sufficient length of time. The hardness of the water affects only the length of the period between regenerations; it is not a factor in determining the rate of filtration, but, of course, a low rate should be used for hard waters to maintain the length of the period between regenerations.

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[1] See also POWELL, S. T., "Trends in Zeolite Softening," *Jour. A.W.W.A.*, November, 1937, p. 1722.

Two or more softeners should be installed in order that one may be producing softened water when the other is being regenerated; or where a single softener is installed, a by-pass should be provided for use during the time that the softener is out of service and for emergency use. The time required for regeneration should not exceed an hour and may be as short as 20 min.

610. Regeneration with Salt. The point at which the softener should be regenerated may be determined by a soap test of the hardness of the effluent, or it may be regenerated at regular intervals regardless of the performance. In some cases automatic devices have been installed for the regneration[1] of softening units.

When the time comes to regenerate, the first step is to backwash the softener in a manner similar to a rapid sand filter, to loosen the particles and remove any material that might have been deposited on the bed. The proper amount of 5 to 10 per cent brine solution is then either introduced into the bed through the perforated underdrainage system or run on to the surface of the bed where it is allowed to stand in contact with the entire bed of zeolite during the regeneration period. The brine is then flushed out and the unit put back into service, but the effluent from it should be wasted until it shows a hardness of less than 1 p.p.m.

The softener is not satisfactorily regenerated unless the first 10 per cent of the water softened between regenerations shows a chloride content of less than 10 p.p.m. higher than that in the unsoftened water and no increase in the chloride content of the water during the remainder of a run between regenerations.

The salt used for regeneration should contain at least 96 per cent NaCl and should be free from iron, mud, chips, and other foreign matter. The amount of salt required depends on the volume of zeolite present, its exchange capacity, and the assumed allowance of salt per unit of hardness removed. As salt is the most important item in the cost of zeolite softening, every effort should be bent toward the utilization of the minimum possible amount. In general, experience has shown that about ½ lb. of salt is used per thousand grains of hardness removed, the minimum claimed being 0.2 lb. per thousand grains.[2]

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[1] See also APPLEBAUM, S. P., "Development of the Automatic Zeolite Water Softener," *Jour. A W.W.A.*, 1934, p. 607.

[2] See also POWELL, S. T., "Trends in Zeolite Softening," *Jour. A W.W.A.*, November, 1937, p 1733.

Example: Let it be assumed that the unit contains 1,000 cu. ft. of zeolite and that it has a capacity, in terms of $CaCO_3$ removable, of 3,000 grains per cu. ft. The unit will have, therefore, a softening capacity of 3,000,000 grains. If it is assumed that 0.5 lb. of salt is required per 1,000 grains of hardness removed, there will be required 3,000 × 0.5, or 1,500, lb. of salt to regenerate the unit. Each gallon of saturated brine solution contains 2.479 lb. of salt. Approximately 605 gal. of saturated brine solution will be required, which should be diluted to 1,740 to 3,480 gal. to form a 5 to 10 per cent solution.

In greensand softeners the maximum efficiency is obtained when approximately 1.4 lb. of salt is used per cubic foot of zeolite, and the use of a larger amount of salt will cause little increase in capacity. In no case should the amount exceed 6 lb. per cu. ft. of zeolite.

Practice is tending toward the storage of salt in "wet storage," *i.e.*, as a saturated brine solution, containing about 25 per cent sodium chloride, in concrete tanks painted with salt-resistant paint, such as Inertol or Acron. The more frequent the regeneration, the smaller the softener, the lower its first cost, the greater the attention required in operation, and the sooner the complete loss of softening capacity of the zeolite. Automatic water softeners for municipalities and industrial plants have been designed for regeneration every two hours. Some domestic softeners may run for months between regenerations. The most desirable frequency of regeneration cannot be fixed definitely, because the factors involved are not sufficiently well fixed by practice and experience.

611. Loss of Softening Capacity. Loss of softening capacity may be due to such factors as (1) the physical and chemical conditions of the grains of the zeolite, (2) the temperature of the raw water, (3) the rate at which water is passed through the zeolite, (4) the amount of salt used, and (5) the uniformity of distribution of brine during regeneration. No deleterious effect on the zeolite seems to result from its operation to exhaustion between regenerations.

612. Washing Equipment. In designing the washing equipment for an upward-flow wash, a rate of 6 to 8 g.p.m. per sq. ft. of zeolite surface will expand synthetic zeolites about 50 per cent, and a 6-gal. rate through greensand will expand it about 25 per cent. A rate higher than about 2 to 3 cu. ft. per sq. ft. per min. may result in disturbing the gravel layer, causing an undesirable mixture with the zeolite unless provision is made for holding the gravel in place. The wash-water gutters should be high enough above the surface of the bed to permit at least 50 per cent expansion without loss of zeolite. In designing the underdrainage system the provision of $^3/_{16}$-in. holes on 6-in. centers will provide sufficient area, giving a loss of head of about 8 ft. when washing at an 8-gal. rate.

613. Control of Municipal Zeolite Softening. Water softened to zero hardness by zeolite action requires adjustment of its pH value and calcium carbonate content to be noncorrosive to iron.[1] The adjustment is important, because undersaturation with calcium carbonate results in corrosion, and supersaturation builds up scale.

The corrosiveness of unadjusted hot softened water may be so great that pipes and equipment can be quickly ruined unless the aggressiveness of the water is neutralized. This fact cannot be too greatly stressed. The ignorant use of zeolites in softening hot water may be disastrous.

Since it is uneconomical and unnecessary to produce a water of zero hardness for ordinary municipal use, it is customary to soften only a portion of the water supply to zero hardness and to mix it with the proper quantity of unsoftened water to produce the desired hardness in the finished product.

614. Comparison of Lime-Soda and Zeolite Processes. The two principal processes for municipal water softening are the lime-soda and the zeolite process. There are certain conditions under which one may be more desirable than the other, and circumstances may arise justifying a combination of the two. A list of the advantages sometimes attributed to each process follows:

Advantages of the Lime-Soda Process

1. Suitable for turbid, chalybeate, and acid waters where zeolite cannot be used.
2. The effluent contains lower total solids than that from zeolite softening.
3. Control of corrosion prevention is comparatively simple.
4. May be more economical.
5. Has bactericidal effect.
6. May be better for excessively hard waters, particularly those high in magnesium hardness, and for water high in sodium.
7. Easily added to an existing filter plant.
8. Freedom from patent complications.
9. Freedom from danger of loss or disintegration of material.

Advantages of the Zeolite Process

1. Freedom from sludge.
2. First cost and operating cost are comparatively low.
3. Free from danger of excess chemicals in the effluent.
4. Independent of change in quality of raw water.
5. No deposits in the distribution system.
6. The chemicals involved are easy to handle.
7. Highly skilled labor is unnecessary.
8. Any desired reduction of hardness is attainable.
9. Repumping is unnecessary by the use of pressure softener.
10. Small space is required.
11. Almost completely automatic.

SYNTHETIC ORGANIC DETERGENTS

615. Other Substances Used. Synthetic organic detergents, the so-called sulphonated oil soaps and related compounds, and proprietary products usually containing organic compounds are used in the processing of industrial water supplies and for domestic purposes, but are not used for the softening of public water supplies. The sulphonated compounds lather in hard water without producing a precipitate. They include sodium metaphosphate, $NaPO_3$, sodium hexametaphosphate $(NaPO_3)_6$, called Calgon[1] and sodium pyrophosphate, $Na_4P_2O_7$. In high-pressure steam boilers sodium phosphate or certain organic compounds may be used to inhibit scale formation. These compounds include lignin sulphonate, concentrated sulphite waste liquor, and a host of proprietary compounds for some of which great claims are made.

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[1] See also BAYLIS, J. R., "Treatment of Water to Prevent Corrosion," *Ind. Eng. Chem.*, July, 1927, p. 777.

Sodium hexametaphosphate, or Calgon, is a proprietary compound. It is a highly soluble hygroscopic chemical, added in about 25 per cent solution, in a concentration of about 1 p.p.m. Close control is not essential.[2] The cost of dosing at this rate is about $1.20 per million gallons.

These substances are used also to control the corrosive aggressiveness of water, as explained in Sec. 659.

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[1] See also JONES, K. K., "The Physiology of Sodium Hexametaphosphate," *Jour. A W W.A.*, September, 1940, p. 1471.

[2] See also RICE, OWEN, "The Advantages of Metaphosphate over Carbon Dioxide for Stabilization of Lime-softened Water," *Southwest Water Works Jour.*, Vol. 22, p. 21, April, 1940.

CHAPTER XXX

DISINFECTION

616. Purpose and Methods. Water is disinfected to kill bacteria and thus prevent water-borne disease. Disinfection is not a substitute for filtration. It does not improve the physical characteristics of the water. It may have either no effect or a detrimental effect thereon. Chlorination, the most common method of disinfection, frequently causes an increase in taste and odor. Disinfection usually requires complicated mechanisms that need the attention of skilled operators to avoid breakdown and incorrect dosage. The dangers from the interruption of service are so great that they must be avoided by every means possible.

Bacteria in water may be killed by chlorination, the addition of ozone, exposure to the ultraviolet ray, heating, the addition of lime, and in other ways. All microscopic organisms and larger forms of life in water may not be removed by all these methods; *e.g.*, *Endamoeba histolytica* are not removed by ordinary chlorination. Sterilization by heating is suitable only for household use. The expense involved in heating and cooling and the undesirable tastes resulting therefrom preclude this method from use for large water supplies. Sterilization with lime alone has not been extensively practiced. It has been found that *E. coli* will die in water with a pH greater than 9.5. Hence, when enough lime is added to bring the pH to this figure, sterilization of *E. coli* occurs.

617. Chlorination. The use of chlorine has become practically universal in the disinfection of water supplies taken from questionable sources. Knowledge of the mechanism of the effect of chlorine in killing microorganisms is needed to explain its selective action that results in its greater effectiveness as a bactericide than in killing algae, protozoa, and other microorganisms. It is widely believed that the bactericidal power of chlorine is dependent on the formation of un-ionized hypochlorous acid, HOCl, and that the effect is directly proportional to the concentration of un-ionized hypochlorous acid present. It follows, therefore, that the application of chlorine to an alkaline solution with high pH will be more effective than to an acid solution with low pH since a greater concentration of the un-ionized acid will be released at the low pH. For equal concentration of chlorine added, other conditions being equal, 150 times as much chlorine must be added at a pH of 5 than at a pH of 10 to produce the same killing effect. Other conditions that may affect the bactericidal power of chlorine include organic matter and turbidity.[1] Organic matter and turbidity react with the chlorine to neutralize its effect and prevent the formation of hypochlorous acid. Within the normal range of temperature of potable water, temperature has little effect on chlorine activity. The concentration of chlorine added is highly important but, with break-point chlorination control, considerable variance is found in chlorine residuals that cause consumer complaint.

Hypotheses that have been advanced to explain the bactericidal effect of chlorine include (1) the effect of *nascent oxygen,* which assumes that the decomposition of HOCl releases *nascent oxygen* that oxidizes the organic matter of which the bacteria are composed; (2) the effect of direct chlorination resulting from the reaction of free chlorine with the bacterial protoplasm; and (3) the fact that toxic substances are formed by the reaction of the chlorine with substances in the bacterial cell wall. These interfere with further cell division by killing the organism. None of these hypotheses fully explains the lethal effect of chlorine on microorganisms.

Although chlorine is used principally for the killing of bacteria, it may be applied to water also to aid in the removal of iron or manganese, for the cleaning of sand filters, for the sterilization of water mains, and for other purposes.

Chlorine may be applied to water with equal effectiveness in the form of bleaching powder or as a gas, or it may be formed in the water by the electrolysis of chloride solutions. Its effectiveness as a bactericide does not depend on the method of its application. Conditions that may affect its results include the quality of the water and the period of contact permitted. For example, alkaline waters are more effectively chlorinated than acid waters, and bactericidal action will usually not take place until the chlorine has reacted with all organic matter susceptible to chlorination. The addition of chlorine to polluted water is, therefore, inefficient and expensive and may be disastrous owing to the creation of tastes and odors.

618. Effect of Chlorine on *E. histolytica.* In order to find a sterilizing agent capable of destroying cysts of *E. histolytica,* which are not satisfactorily destroyed by chlorine alone, studies have been made[2] of the cyst-penetrating power of chlorine and chloramine compounds in water. These compounds are as follows, arranged in relative order of activity:

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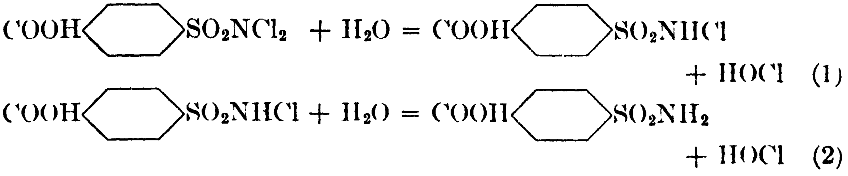
[1] See also COSTIGAN, S M., "A Bacteriologist Looks at Chlorine," *Jour A.W.W.A.*, March, 1942, p 353

[2] See also CHANG, SHIH LU, "Destruction of Micro-organisms," *Jour. A W.W.A.*, November, 1944, p. 1192.

Chlorine, Cl_2
Hypochlorous acid, HOCl
Halazone[1]
Dichloramine, $NHCl_2$
Succinchloramide
Monochloramine, NH_2Cl

All these substances, except chlorine and hypochlorous acid, are chloramine compounds; halozone and succinchloramide include sulphur.[1] In the presence of water they break down to produce HOCl, as indicated by the following reactions:



The very important effect of pH on the bactericidal and cyst-penetrating power of chlorine is to be noted because it may greatly affect the concentrations of chlorine required for satisfactory disinfection. It is generally assumed that the disinfecting effect of chlorine is proportional to the concentration of undissociated hypochlorous acid present and that this weak acid is but slightly ionized in acid solution but almost completely ionized in basic solution. These facts may help to explain why it takes 300 times as many units of titrable chlorine at pH 11 than at pH 5.0 to have an appreciable cysticidal effect on *E. histolytica*.[2]

619. Amount of Chlorine Required. The amount of chlorine required is dependent on the organic content of the water to be treated, its hydrogen-ion content, the amount of carbon dioxide present, its temperature, the time of contact, and other factors as well as the desired results. In practice it is customary to apply the chlorine until the residual found in the treated water ready for the consumer is between a trace and about 0.1 to 0.2 p.p.m. The amount of chlorine required in practice to produce this result is between a trace and 1 p.p.m.

620. Bleaching Powder and Quicklime. Bleaching powder, or calcium hypochlorite, $CaOCl_2$, is a chlorinated lime containing, when freshly made, about 33 per cent, by weight, of chlorine available for sterilization. The material was used exclusively when chlorination of water supplies was first introduced. It is now rarely used in the sterilization of public supplies, having been superseded by liquid chlorine. H-T-H (high-test hypochlorite) is a stable, easily soluble, free-flowing, nonhygroscopic powder carrying 65 per cent available chlorine. It overcomes many of the disadvantages found in the use of bleaching powder, and its cost, per unit of available chlorine, is no greater. Perchloron is a proprietary product containing slightly less than 70 per cent of available chlorine. None of these powders is used to any extent in the chlorination of public water supplies. They are found more often in the chlorination of water mains, filter sands, and storage reservoirs and under emergency conditions where liquid chlorinators are unavailable or unsuitable.[1]

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[1] See also DAKIN, H. D., and E. K. DUNHAM, "The Disinfection of Drinking Water," *Brit. Med. Jour.*, May 26, 1917, p. 682; Halazone, *ibid*, is *p*-sulphondichloramidobenzoic acid.

[2] See also BRADY, F. J, and others, "Effect of Chlorination of Water on Viability of Cysts of Endamoeba Histolytica," *War Med.*, April, 1943, p. 409.

It has been found that the application of quicklime, CaO, to water in doses of about 12 grains per gal. has a distinct bactericidal effect.

621. Handling Chlorine. Gaseous chlorine is a greenish-yellow poisonous substance, with a typical odor, and is about 2.5 times heavier than air. Liquid chlorine is an amber-colored oily liquid about 1.5 times heavier than water. A unit volume of liquid at 0°C. and a pressure of 776 mm. of mercury gives 462 units of gas. The gas will neither burn nor explode. In the presence of water or water vapor it is extremely active chemically and will combine rapidly with most metals. The dry gas is extremely corrosive to metals at temperatures above 300°F., and it may become corrosive at 195°F. Commercial liquid chlorine is anhydrous and about 99.5 per cent pure.

The dry gas and liquid chlorine can be stored safely and indefinitely in steel cylinders, under proper conditions. Cylinders are constructed under specifications of the Interstate Commerce Commission to hold from 100 lb. of chlorine to more than a ton for the largest containers. The cylinders are usually filled to 80 per cent of their capacity with liquid chlorine at 68°F., although they may legally be filled to 88 per cent capacity at 70°F. They are built to withstand a bursting pressure of about 500 p.s.i., corresponding to a temperature of about 190°F. Cylinders should be kept below 150°F., should be stored upright, and should be carefully handled, rolling or dropping being dangerous.

Care must be taken to prevent the escape of liquid chlorine, because of its highly toxic nature except in high dilution. It is desirable to have gas masks in good condition available wherever chlorine is stored or used. Minor leaks in chlorine apparatus can be detected by the use of ammonia fumes which combine with the chlorine to form dense white clouds of ammonium chloride. Since chlorine gas is heavier than air it will accumulate first in low places and near the floor.

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[1] See also KIENLE, J. A., "Where Dry Hypochlorites Pay in Water Works Practice," *Water Works & Sewerage*, October, 1932, p. 359

Dangerous compounds may be formed when chlorine comes in contact with other gases. For example, the escape of chlorine into a poorly ventilated space containing carbon monoxide will result in the formation of phosgene, a highly toxic gas used frequently during the First World War. In the use of ammonia in connection with chlorine, in the chloramine process, care must be taken not to mix the gases before they are applied to the water, because of the danger of forming the highly explosive substance nitrogen trichloride.

Steel burns brightly in dry chlorine, with great evolution of heat, at temperatures above 194°F. It is essential, therefore, that the chlorine be kept in a cool well-ventilated room until ready for use. The temperature in the dosing room may be raised to 100 or 110°F. to enhance the volatilization of the liquid. Liquefaction of chlorine in the chlorinator or in cooler portions of the pipe line carrying chlorine will result if the containers are stored at higher temperatures. The chlorine gas, if dry, may be conveyed to the point of application in pipes of any serviceable material, but, if moist, rubber, glass, or silver may be used. However, neither hard nor soft rubber may be used with liquid chlorine. The pipe should be at least $^3/_4$ in. in diameter and arranged to drain back without traps into the container. Otherwise drip legs may be provided at low points to collect any condensate. If the room is insufficiently heated the upper portion of the drip traps may be insulated and the lower portion kept warm by a 100-watt electric light bulb.

The ventilation of the room should admit air, under forced draft, near the ceiling and permit the escape of chlorine at the floor level. Steam eductors are highly satisfactory for this purpose, as the steam quickly absorbs the chlorine, solving the disposal problem.

Some recommendations made by a committee of the American Water Works Association[1] for the handling of chlorine include the following: (1) Never connect a full container on to a header with other containers until temperatures (pressures) of both are approximately the same. (2) Never apply flame or blow torch to a container. (3) Keep valves closed . . . except when delivering chlorine.

622. Application of Chlorine. Chlorine gas may be fed directly to the point of application to the water supply, or, preferably, the gas may first be dissolved in a small flow of water and the solution fed to the point of application. Direct application of chlorine to the water to be treated is less satisfactory because of unsatisfactory diffusion, possibility of the concentration of chlorine around the diffuser at low temperatures, and possibility of corrosion in pipes and valves resulting from accumulation of undissolved gas. On the other hand direct application is less expensive than solution-feed application.

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[1] American Water Works Association, "Water Quality and Treatment Manual," 1940, p 141.

623. Application of Liquid Chlorine. Chlorine is most effective when applied to filtered water at such a point that adequate mixing is assured and a period of contact of not less than 10 to 15 min. is provided between the chlorine and the water before the chlorinated water reaches the user. The application of chlorine to a reservoir or a suction line is usually to be preferred to application to a pipe under internal pressure. Where chlorine is applied under pressure, it is found to be as effective but not so easily handled. The use of direct-feed chlorinators is limited to pressures of 25 to 30 p.s.i. in the delivery pipe, whereas solution-feed equipment will carry to any pressure.

Provision is usually made to mix the chlorine and the water immediately after application and during the period of reaction. This provision need not be elaborate, the mixing and the time in the distributing mains frequently being adequate. Care should be taken in the design of a distribution system that water cannot be drawn from the mains in a shorter time than about 15 min. after the application of the chlorine because of possible ineffectiveness of the chlorination in a shorter time or of the receipt of overdoses of chlorine.

624. Liquid Chlorinators. Reliable and satisfactory apparatus for the application of chlorine to water are available and are in extensive use. One form is illustrated in Fig. 255.

Care is necessary to keep such machines and the lines leading to them free from dirt and deposits to avoid fluctuations in their operation and other difficulties. Gummy deposits of material collected from the lines by the chlorine occur in the orifices or on the bell jar. Such material may come from the pipe linings, oil used in valve packing, etc., and from unsatisfactory cleaning materials. Only the methane derivatives such as chloroform, wood alcohol, or carbon tetrachloride are satisfactory for cleaning chlorine equipment, as they are not converted into troublesome solids by the chlorine.[1]

625. Determination of Residual Chlorine. It is desirable to know something of the method of determination of residual chlorine before a conclusion can be safely based on the results reported. Five methods are recognized by a Committee of the American Water Works Association.[2]

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[1] See also Cox, C. R., "Equipment for the Chlorination of Small Water Supplies," *Jour. A.W.W.A.*, 1934, p. 1587.

[2] Committee of American Water Works Association, *Jour. A.W.W.A.*, October, 1943, p. 1315.

The orthotolidine method is probably most widely used. However, the method is not foolproof as substances in the water may interfere with the results obtained, and impurities in the chemicals used in making up the solutions may likewise affect the result. Interfering substances include nitrite, ferric compounds, manganic compounds, organic iron compounds, lignocellulose, and algae. The effect of these substances is to increase the apparent content of chlorine. Suspended matter, turbidity, and color may affect the test results. It is apparent that the standard orthotolodine test for chlorine must be conducted under well-controlled and understood conditions in order to avoid error in the interpretation of its results.

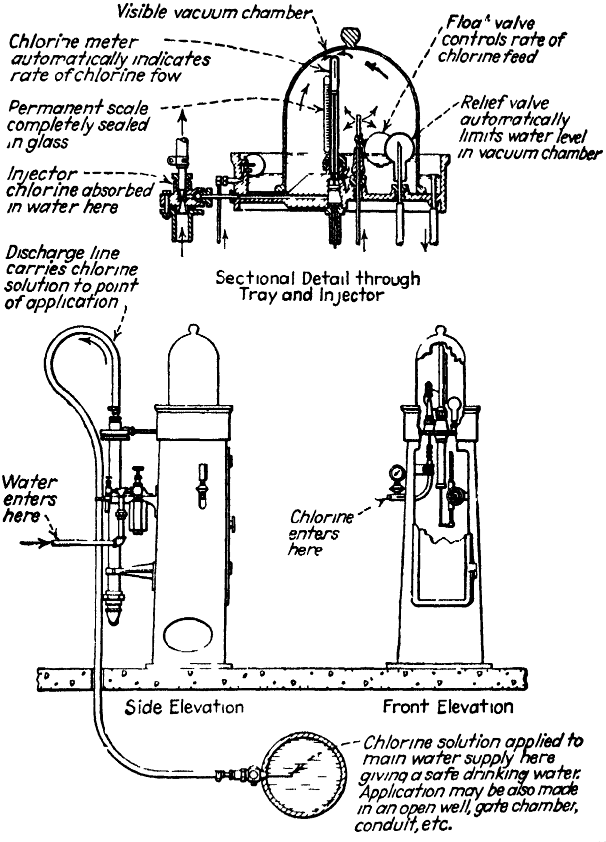
Visible vacuum chamber
Chlorine meter automatically indicates rate of chlorine fow
Float valve controls rate of chlorine feed
Permanent scale completely sealed in glass
Relief valve automatically limits water level in vacuum chamber
Injector chlorine absorbed in water here
Discharge line carries chlorine solution to point of application
Sectional Detail through Tray and Injector
Water enters here
Chlorine enters here
Side Elevation
Front Elevation
Chlorine solution applied to main water supply here giving a safe drinking water. Application may be also made in an open well, gate chamber, conduit, etc.

FIG. 255.—Manual-control chlorinator solution-feed type (*Courtesy of Wallace and Tiernan.*)

The iodometric method can be made more precise than the orthotolodine method, particularly when residual chlorine is greater than 1 p.p.m. Colorimetric methods for the iodometric determination of chlorine are available.

The amperometric method is the only procedure known that will differentiate between free chlorine and free chlorine combined with nitrogenous compounds. The test is known as the Laux "flash" test. This is a colorimetric test in which the color must be standardized. It is rendered inapplicable in the presence of oxidized manganese.

The chlorine-demand test involves the determination of the chlorine demand of a water by either the orthotolidine test or amperometric methods. It is of use mainly in the control of the break-point methods of chlorination control.

The drop-dilution test is a rapid field method applicable especially where chlorine concentrations are greater than 10 p.p.m. as in water-main sterilization. The method is a modification of the orthotolidine test, using orthotolidine as a reagent. The chlorine-demand test for field use, depending on orthotolodine reagent, gives only rough approximations of the true chlorine demand.

626. Tastes and Odors Due to Chlorine. The addition of chlorine to water may result in an unpleasant taste, due to the presence of too much residual chlorine or to a combination between the chlorine and matter in the water. Tastes may be avoided by adding a larger amount of chlorine than is required for sterilization alone, called "superchlorination"; by superchlorination followed by aeration; by superchlorination, a long period of contact, followed by dechlorination; by chlorination before and after filtration, called "double chlorination"; by chlorination preceding filtration; by chlorination after filtration; by the use of ammonia in conjunction with prechlorination or double chlorination; by lowering the pH of the water to assure a more rapid dissipation of the residual chlorine; by passing the water after chlorination through a filter of activated carbon; or by various combinations of these methods.[1]

Chlorophenol tastes are the most difficult to combat because of the very faint traces of phenol that will produce noticeable results. Tests made by Howard[2] at Toronto showed that iodoform tastes were observed in the filtered water with a phenol content of 0.006 p.p.m. after chlorination with 0.2 p.p.m. of chlorine. Authorities differ as to the concentration of phenol necessary to produce tastes in water when chlorinated.

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[1] See also Howard, N. J., "Modern Practice in the Removal of Taste and Odor," *Jour. A.W.W.A.*, Vol. 9, p. 766, 1922; and Enslow, J. H., *Jour. A.W.W.A.*, Vol. 18, p. 621, 1927.

[2] Donaldson, W., and R. W. Furman, "Quantitative Studies in Water Supplies," *Jour. A.W W.A.*, Vol. 18, p. 605, 1927.

Thresh, in the *Nineteenth Report of the Metropolitan (London) Water Board*, reported that as little as 0.0002 p.p.m. of phenol with 0.25 p.p.m. of chlorine produced a taste.

The presence of 0.005 p.p.m. of iodine in water may result in iodoform tastes after chlorination. Superchlorination and oxidation with potassium permanganate may overcome them, but dechlorination with sulphur dioxide has no effect. The use of activated carbons is reported by Ellms to have beneficial effects in removing such tastes.

Difficulties have been encountered with tastes due to chlorinated water that has been in contact with coal-tar pipe coatings.

627. Prechlorination, Double Chlorination, and Superchlorination. These have been defined on page 530. It has been found that double chlorination is advantageous under some conditions. Prechlorination will reduce the bacterial load on the filters so as to minimize the possibility of the passage of bacteria into the filtered water, without diminishing the efficiency of filtration. The amount of coagulant required may also be reduced because of the oxidation of the organic matter by prechlorination. Advantages claimed for prechlorination include[1] (1) reduction of bacterial load on the filters, (2) increased factor of safety, (3) better color removal in certain instances, (4) increased filter runs, (5) control of plankton in basins and filters, (6) deferred putrefaction of settling-basin sludge, and (7) elimination of tastes and odors. Despite the advantages of prechlorination, postchlorination should not be abandoned, because of the safety barrier interposed by it.

Double, or multiple, chlorination refers to the application of chlorine at two or more points in the purification process. Among the advantages of double chlorination are: the lightening of the load on the filters; the maintenance of two chlorinating plants serves as a factor of safety; the exposure of bacteria to high concentrations of chlorine for short periods of time is more effective than exposure to small concentrations for a long period of time; the long period of retention in the coagulating basin permits relatively large overdoses of chlorine before free chlorine appears in the filter effluents; tastes and odors are reduced; algae and slime are avoided in coagulating basin and filters; coagulation is aided.[2]

Superchlorination is occasionally used to destroy odors and tastes. It has been used to destroy the taste resulting from chloro- products formed between the decomposition products from vegetable matter and algae in the raw water and in the coagulated water. Raw-water chlorination, preceded or followed by the application of activated carbon, is a widely followed practice to destroy tastes and odors. The excess chlorine may be applied at any point, or points, in the purification process. After filtration is the time at which it is most frequently applied. The excess chlorine must thereafter be removed to avoid chlorine tastes in the water.

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[1] NORCOM, G. D., "Control of Odor Marks Progress in Sterilization," *Eng. News-Record*, June 8, 1933, p. 745.

[2] Cox, C. R., *Jour. A.W.W.A.*, Vol. 16, p. 55, 1926.

The best point, or points, and the desirable amount of chlorine to apply at each point are determined by trial. In general, the minimum amount of chlorine necessary to sterilize water will be required by the postfiltration application of the chlorine. At this time there is the least amount of organic matter in the water to consume the chlorine, its effect being concentrated on the remaining bacteria. Although the amount of chlorine required is increased by its application at other points, the results of purification may be improved.[1]

628. Dechlorination. The purpose of dechlorination is to remove excess chlorine from the water before distribution to the consumer to avoid chlorine tastes. Dechlorination may be accomplished by applying sodium thiosulphate, sodium bisulphite, or sulphur dioxide, as a gas or in solution, to the water. The sulphur dioxide gas is applied in much the same manner as chlorine is applied, with a contact period for dechlorination of not less than 10 to 15 min. Experience has shown that 0.3 to 0.6 p.p.m. of sulphur dioxide may be required for success with dechlorination. The relationship between sulphur dioxide and an excess of chlorine is about 1.0 part of chlorine to 1.12 parts of sulpur dioxide. This is about 25 per cent in excess of the theoretical requirements. Dechlorination may also be accomplished by treating the water with magnesium metal,[2] by prolonged storage of the water particularly when exposed to sunlight, by aeration, and by the use of activated carbon.

Potassium permanganate may be used with success for the removal of chlorine tastes. The procedure calls for its addition to the water in concentrations less than 0.3 p.p.m. The potassium permanganate is applied to oxidize the organic matter preceding chlorination. When it is used, care must be exercised to correct for the effect of manganese on the orthotolidine test for residual chlorine.

629. Break-point Chlorination. If chlorine is added to water that has no chlorine demand the concentration of residual chlorine should equal the concentration of applied chlorine, as indicated by line *A* in Fig. 256. If, however, the water does have some chlorine demand and chlorine is added in ever-increasing concentration, the relation between applied and residual chlorine may appear as indicated by line *B*. Point *D* on line *B* is called the "break point." The break point in the chlorination of water may be defined as the point on the applied-residual chlorine curve at which all, or nearly all, the residual is free chlorine.

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[1] "Double Chlorination at Quincy, Illinois," *Eng. News-Record*, Vol. 100, p. 407, 1928.

[2] See also Bach, H., "Treating Water with the Aid of Magnesium Metal," *Water Works & Sewerage*, July, 1934, p. 241.

The shape of line *B* results from the fact that when chlorine is first applied the immediate chlorine demand results in the formation of stable chlorides and some unstable chloramines. The latter give color in the conventional orthotolodine test and are reported as residual chlorine. At point *C* on the curve the increasing concentration of the applied chlorine breaks down the chloramines that have been recorded as residual chlorine, and the true residual free chlorine is revealed at point *D* where chlorine demand has been satisfied.

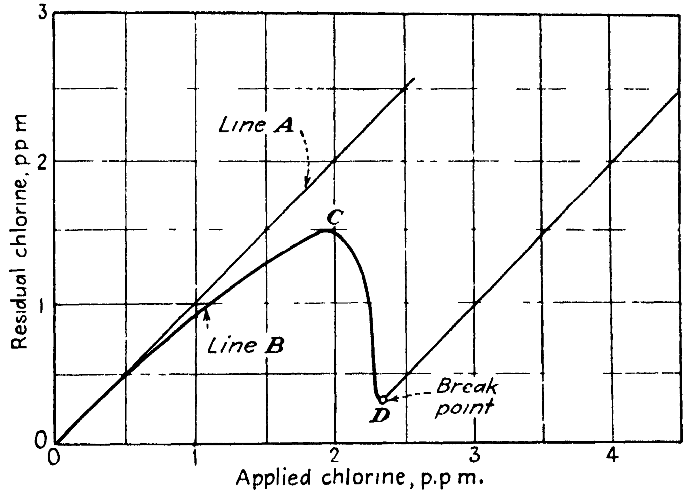

FIG. 256. Break-point chlorination.

It has been found by experience that the application of chlorine at, or slightly higher than, break-point concentration will remove tastes and odors, will have adequate bactericidal effect, and will leave a desired chlorine residual. Unfortunately the universal adoption of the method is made difficult by the fact that all waters do not present a distinguishable break point, and in some the changes in the quality of the raw water may effect rapid changes in the break point. The break point is caused by the presence of nitrogen compounds in the water. The break point is distinct with ammonium sulphate, but when nitrogen is in the form of amino-acetic acid it is quite indistinct. In some cases a recognizable break point has been induced by the addition of ammonia as NH_3, NH_4OH, or $(NH_4)_2SO_4$.

Before attempting sterilization by break-point chlorination, laboratory tests should be made to show that the process is practical on that particular water.

630. Chlorine-Ammonia Treatment. The application of ammonia[1] to water prior to the addition of chlorine has the advantages of (1) the prevention of tastes, particularly those due to phenols; (2) the control of microorganisms in settling basins, filters, and the distribution system, due to the possibility of dosing heavily with chlorine and the carrying of high chlorine residuals without the production of tastes; (3) stronger bactericidal effects than with chlorine alone where considerable quantities of organic matter are present; (4) longer period of persistence of the bactericidal effect, which inhibits aftergrowths; (5) reduction of chlorine requirements; (6) a reducing action which renders its effects independent of the presence of organic matter; (7) less irritation of the eyes and nose, resulting in its use in swimming pools; (8) operators, no longer fearing to overdose with chlorine, will feed an adequate dose; and (9) freedom from danger, since chloramines are nontoxic and involve no danger either to employees or to consumers.

Abstracts from conclusions made by a committee of the American Water Works Association[2] are as follows: (1) The process is a means for disinfecting water where chlorinous tastes and odors must be limited. (2) The method will maintain residual chlorine values in distribution systems and other parts of the water supply. (3) The treatment should be adopted only after complete investigation of its efficacy under local conditions. (4) The treatment must be carefully controlled, residuals maintained must not be less than twice those found effective with chlorine alone, and a minimum period of contact of 2 hr. must be provided. (5) When taste control is not a factor, chlorine may be added in advance of ammonia to obtain rapid disinfection followed by the application of only sufficient ammonia to react with the chlorine to maintain adequate residual chlorine concentrations in the distribution system. (6) The orthotolidine test should be used to measure chloramine residuals. (7) The control of algae by the use of copper sulphate in conjunction with chlorine and ammonia is no more effective than the use of copper sulphate alone.

It is important that the ammonia be applied at the right point and that adequate mixing and time of contact be provided before the application of the chlorine. This time should be between 20 min. and 1 to 2 hr. The ammonia may be applied at any convenient point, an open reservoir or a closed conduit, where the efficacy of application has been proved by experience. Because the ammonia dissolves quickly in the water but does not diffuse readily, it should be mixed by mechanical means. It should be released near the bottom of a reservoir, because its specific gravity is lighter than water. The period of contact may be from a few minutes to a few hours, but it must not be made too long, because the ammonia is a food for nitrifying bacteria which may remove the ammonia before it is available for the action of the chlorine, resulting in an increase in the bacterial and nitrite content of the filtered water.[1] The resistance of these bacteria makes it appear impracticable to control nitrification by chlorination.[2] If insufficient chlorine is added to carry a residual through the filters, the addition of more chlorine will probably cause tastes, as chloramines do not remove phenols from water, and the postaddition of chlorine, unprotected by ammonia, will cause phenol tastes.

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[1] See also GRIFFIN, A. E., *Am. Jour. Pub. Health*, March, 1937, p. 226.

[2] Committee American Water Works Association, *Jour. A.W.W.A.*, December, 1941, p. 2079.

It has been found that the application of approximately 0.25 p.p.m. of ammonia, NH_3, is sufficient in most cases to prevent taste production when followed by chlorination. The theoretical ratio of chlorine to ammonia is 4:1 at pH 8.5 and 8:1 at pH 4.5. Ratios should vary with pH values. In practice it has been found that ratios of 3:1 to 2:1 will produce satisfactory results.

Ammonia gas is highly soluble in cool water, making its addition to the water easy. It can be applied through devices similar to those used for the application of liquid chlorine. The ammonia water should be fed directly to the public water supply through an orifice box. The cost of this treatment has been found to be about 40 to 60 cents per million gallons of water treated.[3]

631. Handling Ammonia. Care must be taken in the handling of liquid ammonia, as in the handling of liquid chlorine Both may be dangerous to life and destructive of property if allowed to escape from their containers. A mixture of ammonia and air in the ratio of 1:5 is inflammable. When moist, ammonia is corrosive to metals, is highly irritating to the mucous membranes, and is especially corrosive to copper and copper-bearing alloys. Ammonia containers should be stored under conditions similar to those recommended for the storage of chlorine containers.

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[1] See also HULBERT, R., "Ammonia-chlorine Treatment Yields Nitrites," *Eng. News-Record*, Sept. 4, 1933, p. 315.

[2] See also FEBEN, D., "Nitrifying Bacteria," *Jour. A.W.W.A.*, April, 1935, p. 439.

[3] See also McAMIS, J. W., *Jour. A.W.W.A.*, Vol. 17, p. 341, 1927; SPAULDING, C. H., *Jour. A.W.W.A.*, Vol. 21, p. 1085, 1929; HARRISON, L. B., *Jour. A.W.W.A.*, Vol. 21, p. 542, 1929; JENNE, L. L., and H. R. WELSFORD, *Ind. Eng. Chem.*, Vol. 23, p. 32, 1931; KOSHKIN, M. L., *Jour. A.W.W.A.*, Vol. 27, p. 1477, 1935.

632. Chlorine Dioxide.[1,2] The application of chlorine dioxide, ClO_2, to water for the control of tastes and odors has recently been introduced into water-purification practice. Because of the greater chemical activity of chlorine dioxide than of chlorine in combining with organic matter in water, the disinfection of polluted water with chlorine dioxide may not be economical. Ridenour and Ingols[3] report the bactericidal properties of chlorine dioxide to be as great or greater than chlorine. The action is relatively unaffected by pH values between 6.0 and 10.0. This is important in waters of high alkalinity.

The procedure in the use of chlorine dioxide involves chlorination for disinfection followed by the application of chlorine dioxide to destroy tastes and odors. Coagulation and filtration may precede or follow chlorination and precede the application of chlorine dioxide. Chlorine dioxide is an unstable gas that must be manufactured and used immediately. One recommended method is to pass chlorine gas through sodium chlorite. The reaction forms sodium chloride and chlorine dioxide gas. At room temperature this gas is soluble in water to the extent of 2,900 p.p.m. The dose required for effective taste and odor control has varied in practice between 0.5 and 1.5 p.p.m. of chlorine dioxide.

633. Ozone. Oxygen may exist in the form of molecules containing one, two, or three atoms. A single atom of oxygen is known as "nascent oxygen." It is extremely active chemically. It will exist independently only momentarily, seeking to combine immediately with other atoms of oxygen or with other substances. A molecule containing two atoms is the natural condition of oxygen gas. A molecule containing three atoms is ozone. It is an unstable gas that tends to break down into normal oxygen containing two atoms of oxygen per molecule. In the process of breaking down it releases nascent oxygen which is active in oxidizing the organic matter and in killing bacteria present in water exposed to it.

Advantages claimed for the use of ozone include (1) no disinfecting chemical remains in the water; (2) odors, taste, and color are removed; (3) the process is not expensive. However, the cost is greater than for chlorination, the apparatus required is complicated, and the absence of residual chemical provides no safeguard against subsequent contamination.

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[1] See also SYNAN, J. F., and others, "Chlorine Dioxide A New Development in the Treatment of Potable Water," *Jour. New Engl. Water Works Assoc.*, September, 1944, p. 3; and MACLEAN, W. D., "Treatment with Chlorine Dioxide," *Water and Sewage*, May, 1946, p. 21.

[2] For the use of chlorine dioxide in the removal of tasts and odors, see Sec. 649.

[3] RIDENOUR, G. M., and R. S. INGOLS, "Bactericidal Properties of Chlorine Dioxide," *Jour. A.W.W.A.*, June, 1947, p. 561.

Hann[1] states that the power cost may vary between one and several dollars per million gallons, dependent on power rates and ozone requirements. To generate and apply 1 lb. of ozone will require 14 to 16 kw.-hr. It is economical to apply ozone only to clear filtered water so that the ozone demand may be less than 2 to 3 p.p.m. Adequate disinfection will be obtained with a residual of 0.1 p.p.m. or more.

Ozone is manufactured by the silent discharge of a high-tension electric current passing through a stream of air in an enclosed chamber. It must be mixed intimately with the water to be treated, since its solubility in water is slight and the process is inefficient without the best mixing. The mixing may be accomplished by allowing ozone to bubble up through the water or by allowing the gas to rise through a tower filled with pumice stone down which the water is trickling.

The process has been used more extensively in Europe than in the United States.[2] European experience shows the use of about 40 to 85 watt-hr. per 1,000 gal. of water treated. English experience at Ashton in the treatment of a reservoir water infected with algae and with a swimming-pool water at Scarborough[3] is summarized in Table 89.

TABLE 89. ENGLISH EXPERIENCE WITH OZONE

	Scarborough	Ashton
Air flow, c.f m	933	817
Cycles, alternating current	570	500
Voltage	300 4,250	10,000 20,000
Ozone dose, p p m	0 45 0 8	
Watt-hours per 1,000 gal		50
Watt-hours per gram of O_3	29	29

634. Ultraviolet Ray. The ultraviolet ray offers an effective method for the sterilization of clear water. The light is effective in killing both active bacteria and spores, which are difficult to exterminate by other means. No adequate explanation of the lethal powers of high-frequency light-ray vibrations has been offered. That no toxic substance is produced in the surrounding medium is demonstrated by the survival of organisms protected from the rays in the same medium in which exposed organisms have been killed. The destructive power of light begins in the blue-green region of the spectrum with a wave length of 0.490μ and increases in effectiveness to 0.149μ. In this connection, it is interesting to note that the shortest visible violet rays have a length of 0.410μ and the longest visible red rays a length of 0.810μ.

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[1] HANN, VICTOR, "Ozone Treatment of Water," *Jour. A.W.W.A.*, May, 1943, p. 585.

[2] See also WHITSON, T. B., "The Treatment of Water by Ozone," *Jour. Inst. Civil Eng*, Vol. 21, p. 83, December, 1943.

[3] See also OVERFIELD, H. V., "Ozone Treatment of Water at a Swimming Pool," *Water and Water Eng.*, October, 1943, p. 427.

The ultraviolet-ray machine consists of a mercury-vapor lamp enclosed in a quartz globe. The conditions necessary for the effective operation of the sterilizing machine are that all the water shall flow in a thin clear film close to the sterilizing ray. The water should be well agitated (but not mixed with air), and the exposure to the ray should be continuous.

Ultraviolet lamps commonly use 220-volt direct current and about $3\frac{1}{2}$ amp. The effective penetration of the ray in clear and colorless water is about 12 in.

Ultraviolet-ray sterilizers are used for private water supplies, such as for office buildings, institutions, and particularly swimming pools.

The amount of current consumed is dependent on the type of the machine and whether one or a number of machines are used in series. A single machine can be expected to consume about $\frac{3}{8}$ kw. in passing 120 gal. of water in 1 hr. Two large machines in series consumed 1.54 kw.-hr. while delivering 3,000 gal. of water.[1] At Berea, Ohio, the cost of treatment was $4.52 per million gallons with current at 1 cent per kw.-hr.[2]

635. Oligodynamic Action of Metals. Certain metals, particularly silver[3] when immersed in water have been observed to exert an inhibiting action on bacterial life. The process is in the experimental stage, there being no installations of the type for the sterilization of a public water supply. In this process[4] silver, with or without "activators," such as palladium or gold, is deposited on the surface of sand, porcelain, or filter candles. Water is then passed through the filter or allowed to remain in contact with the silvered surface for a certain time. Although great claims have been made for the advantages of the process, there have been no applications where practical experience might support these claims.[5]

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[1] See also "Water Purification with Ultra-violet Light," *Plumbers' Trade Jour.*, Vol. 73, p. 897, 1923; Vol. 74, p. 36, 1924.

[2] See also SPRINGER, J. F., "Water Purification with Ultra-violet Light," *Pub Works Magazine*, October, 1931, p. 39.

[3] See also GOETZ, A., "Water Sanitation with Silver," *Jour. A.W.W.A*, May, 1943, p. 585.

[4] See also GIBBARD, J. H., "The Treatment of Water with Certain Forms of Silver," *Am. Jour. Pub. Health*, September, 1933, p. 910.

[5] See also MOISEEV, S. V., "The Sterilization of Drinking Water by Silver Coated Sand," *Jour. A.W.W.A.*, February, 1934, p. 217; SUCKLING, E. V., "The Sterilization of Water by Katadyn Silver," *Water and Water Eng.*, Dec. 21, 1931, p. 625; JUST and SZNIOLIS, "Germicidal Properties of Silver," *Jour. A.W.W.A.*, April, 1936, p. 492.

636. Excess Lime. The name "excess lime" has been given to a process of water purification in which sufficient lime is added for softening and clarifying purposes and, in addition, to exert a bactericidal effect. The necessary dose of lime is between 10 and 20 p.p.m. of calcium oxide. After its bactericidal effect has been exerted the excess lime must be removed, preferably by any one of the various methods of recarbonation.[1]

637. Control of Microscopic Organisms. Growth of microscopic organisms in a lake or open reservoir may be controlled in whole or in part by the construction or method of operation of the reservoir; by the application of toxic chemicals, commonly called algicides, such as copper sulphate, chlorine, and chloramine, which are used in public water supplies, and compounds of mercury or other heavy metals, oxidizing agents, acids, caustic alkali, and other toxic substances used in industrial and small water supplies under exceptional circumstances; and by physical methods such as aeration, sedimentation, cutting weed , cleaning the reservoir, or changing the quality of the water.[2]

In the construction of a reservoir to inhibit the growth of certain types of microscopic organisms, shallow reaches and stagnant areas should be avoided. In general, it is undesirable to impound water high in organic matter drawn from a swampy area or to store ground water or treated waters in open reservoirs. In the operation of a reservoir, care should be taken to prevent the seeding of the reservoir by the admission into it of water impregnated with organisms. The admission of microscopic organisms into a water supply may be avoided by drawing the water from depths at which undesirable organisms are not found or by allowing a contaminated reservoir to rest until the crop of undesirable organisms has disappeared. Water drawn from a reservoir may be freed from microscopic organisms by aeration or by chlorination applied to the flowing water.

It is usually preferable to treat infected water continuously at the inlet to a reservoir, a distribution system, or other container than to attempt the destruction of large masses of zoogloeal matter that has accumulated in them. In other words, prevention is easier than removal. Some important considerations in the removal of microscopic organisms are[3] as follows: (1) Certain algal spores can resist any form of algicidal treatment and will later develop on reaching a suitable environment. (2) Where any portion of the raw water is exposed to light for a lengthy period in the purification works, several successive applications of algicide may be necessary to prevent growths and may be preferable to a single application. (3) Applications of algicides should be made at points immediately before the water approaches any known focus of algal development. (4) Certain organisms multiply at certain times of the day, and a concentration of the treatment around such time is frequently most effective. (5) Improved circulation or increased velocity of flow in pipes and channels may markedly favor the development of some organisms throughout the system, although the growth of others may be inhibited in this manner. (6) All microscopic growths are subject to decay and thereby produce soluble organic matter which can pass through any form of filter. (7) In some cases it is desirable to alter the chemical quality of the water to discourage the growth of an organism.

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[1] See also HOUSTON, ALEXANDER, *Munic. Sanit.*, April, 1932, p. 148.

[2] NASON, H. K., "Chemical Methods in Slime and Algae Control," *Jour. A.W. W.A.*, March, 1938, p. 436.

[3] HUSBAND, J. W., "Algal Growths and Water Supply," *Water and Water Eng.*, Vol. 35, p. 765, 1933.

Where organic growths are giving trouble in closed conduits and in covered reservoirs, effective methods of treatment include chlorination, chloramine treatment, the use of copper sulphate, and the cleaning of pipes or reservoirs. These remedies must be applied with care, as the use of chlorine will probably produce tastes and odors, if not properly applied; the use of chloramine serves primarily to delay the action of the chlorine so that it may be more effective in remote portions of the distribution system; cleaning of pipes, if done mechanically, may accelerate the effects of corrosion. In spite of their objectionable features, each of the methods has been used with satisfaction.

638. Use of Copper Sulphate. Copper sulphate is the most frequently used chemical in the removal of microscopical organisms in reservoirs, pipe lines, or distribution systems. Copper sulphate is applied to open water either by placing the crystals in a porous sack and towing it behind a boat moving in a course to distribute the solution immediately below the surface of the water in a uniform manner, or by spraying a solution or sprinkling the powdered crystals over the surface of the water or ice. Copper sulphate is applied to water in closed pipe lines by adding it to the entering water in the form of a solution to apply a concentration of about 0.5 to 0.65 p.p.m.[1]

Results from the treatment of reservoirs by copper sulphate have been so satisfactory that it is used as a standard practice in treating impounded surface waters. The presence of copper sulphate in water in the amounts necessary to remove microorganisms has no detrimental effect on human beings or upon the suitability of the water for a public supply or for most industrial processes. It may, however, have a detrimental effect on some fish, the most susceptible of which are certain species of trout which may be killed by concentrations higher than 0.15 p.p.m. The concentrations of copper sulphate that may kill fish are listed in Table 90.

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[1] See also AMSBARY, F. C., JR., "Progress Report on the Use of Copper Sulphate in the Control of Bacteriological Aftergrowths," *Jour. A.W.W.A.*, March, 1945, p. 294.

TABLE 90.—THE AMOUNT OF COPPER SULPHATE NECESSARY TO KILL FISH*

Species of fish	Copper sulphate, pounds per million gallons	Species of fish	Copper sulphate, pounds per million gallons
Trout	1 2	Pickerel	3 5
Carp	2.8	Goldfish	4 2
Suckers	2.8	Sunfish	11.1
Catfish	3.5	Black bass	16 6

* MAHLIE, W. S., *Jour. A.W.W.A.*, Vol. 10, p. 1,000 1923.

Crystals of copper sulphate, $CuSO_4 \cdot 7H_2O$, are available in sizes from a fine powder to those which will just pass a 1-in. mesh. The crystals are readily soluble in water and can be made up into solutions with a strength of about 25 per cent. Commercial copper sulphate contains about 25 per cent copper. Hence, the application of 0.8 p.p.m. of copper sulphate, or 6.5 lb. per million gallons, will create a copper concentration of 0.2 p.p.m. in otherwise noncopper-bearing water. This is the highest concentration permitted by the U.S. Public Health Service standards for potable waters.

The amount of copper sulphate to be used is dependent on the concentration and type of organism present. Attention is more frequently paid to these two controlling factors, and the water is dosed with more chemicals than is needed to assure the killing of all microorganisms. The amount of copper sulphate necessary for the control of different microorganisms is listed in Table 77.

639. Chlorine as an Algicide. The use of chlorine as an algicide is not extensive, because of the greater satisfaction found with copper sulphate and the difficulty of applying chlorine to a quiescent body of water.[1] Where there is a flow of water through a narrow channel or into or away from a body of surface water or from one body of water to another, chlorine gas can be applied at such a point by the method described in Sec. 623. The method may be used in the control of algae in a distribution reservoir.[2] The amounts of chlorine required to kill algae are listed in Table 77. It is to be noted that sufficient chlorine must be added to accomplish the purpose, followed by dechlorination, if necessary, oi undesirable tastes are likely to be developed.

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[1] See also COHEN, C., "Chlorination for Algae Control," *Jour. A.W.W.A.*, Vol. 17, p. 444, 1926; and SETTE, F. J., "Continuous Algal Control by Chlorination," *Eng. News-Record*, May 28, 1931, p. 885.

[2] See also MANGUM, L. B., "Algae Control by Chlorination at Kansas City, Kansas," *Jour. A.W W.A.*, 1929, p. 44.

640. Control of Weeds.[1] Weeds are undesirable in water-supply reservoirs and open channels because they are unsightly; obstruct the flow of water; cause tastes, odors, and color in the water; and supply food for other undesirable organisms. Time-honored methods for the removal of weeds include oiling, burning, raking, dragging, and chaining. Where possible the reservoir may be lowered and the weeds, after drying, sprayed with oil and burned; or the oil may be used to kill the weeds.[2] However, the use of chemicals is usually less costly and more satisfactory. Their use should be attempted only by those with sufficient knowledge to avoid the great dangers to life involved in the improper use of these toxic substances.

Emergent aquatic weeds, such as cattails and tules, can best be controlled by the use of 2,4-D compound (formed by dichlorphenol, acetic acid, and chloramine) applied at a rate of about 200 gal. per acre, using 1 per cent 2,4-D emulsion. If it is applied by spraying, the solution strength should be about 0.1 to 0.2 per cent. Failures of this treatment have been due largely to neglect to follow with subsequent treatment.

Aquatic weeds, other than emergent weeds, can be controlled by the use of sodium arsenite, copper sulphate, chlorine, and Benoclor (trichlorinated benzines). Since the arsenates are toxic to man and animals they must not be applied in concentrations greater than 0.05 p.p.m. Chlorinated hydrocarbons, such as Benoclor No. 3, probably give the best control. They should be applied at a rate of about 50 gal. per acre and allowed at least 4 days of contact, with precautions to avoid tastes and odors.

641. Control of Small Animal Life. Infestations of public water supplies, by the larvae of flying insects, by crustaceae, and by other small macroscopic animal life have been experienced.[3] The larvae most frequently giving trouble are the aquatic larvae stages of the Chironomidae, commonly called midges. The aquatic period involves the egg, pupal, and larval stages. Eggs may be laid in the water, in a protective gelatinous coating, either anchored or free floating. The larvae resemble an animated letter S or figure 8, are from a few millimeters up to 25 mm. in size, and are called white worms or blood worms according to their size and color. In most cases the larvae will be found embedded in the debris and fine dirt of the reservoir bottom. They breathe and feed without leaving this environment. An entire cycle from egg to adult may occur in 1 to 5 weeks.

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[1] See also GOUDEY, R. F., "Chemical Weed Control," *Jour. A.W.W.A.*, February, 1946, p. 186.

[2] See "Storage Reservoir Troubles," *Water Works Eng.*, June 4, 1941, p. 630.

[3] See also FLENTJE, M. F., "Control and Elimination of Pest Infestations in Public Water Supplies,' *Jour. A.W.W.A* , November, 1945, p. 1194

Control is through mechanical, biological, and chemical means. Mechanical control involves covering the reservoir and screening all openings or the use of excelsior filters, which may be useful also in removing crustaceae. Biological methods include (1) keeping down vegetation near the reservoir, (2) keeping down algal growths, thus removing the food supply, (3) placing predatory fish in the reservoir, and (4) holding the reservoir empty for the needed period of time. Chemical methods involve (1) the use of mineral oil (Acme white oil, viscosity 80 to 85 at 100°F.) to keep air from the eggs, to prevent the pupae from emerging, and to imprison flies alighting for oviposition; and (2) the use of DDT in a concentration of about 0.01 p.p.m. DDT should be used only under experienced supervision. The more common algicides, such as copper sulphate, chlorine, and chloramines, have not given universal satisfaction as larvicides.

CHAPTER XXXI

MISCELLANEOUS METHODS OF WATER PURIFICATION

642. Purposes and Methods. Water is treated primarily to improve its sanitary quality. The most common method of treating it to accomplish this purpose is by means of sand filtration preceded by screening, sedimentation, or coagulation. Disinfection is frequently used independently of or in connection with filtration to improve the sanitary quality of the water. Water may be aerated, chlorinated, or otherwise treated in order to remove tastes and odors. Hard water must be softened to improve its qualities for domestic and industrial purposes. Iron and manganese must be removed to avoid tastes and to prevent the stains produced by these metals. Those methods of treating water to accomplish these purposes which are not explained in preceding chapters include the removal of iron, manganese, and other dissolved metals and gases; the carbonation of water for the removal of tastes and odors; and the treatment of water to prevent corrosion.

643. Iron Removal. Iron is soluble in water only in the ferrous state and occurs frequently as a bicarbonate which is dissoved from peaty soils and from sulphides of iron. It is dissolved by the action of carbonic acid and oxygen taken from the air. Ferric oxide in the rocks through which water flows is reduced to ferrous oxide by the organic matter present in the water and to ferrous bicarbonate due to the presence of carbon dioxide.

Iron in natural waters and in those receiving industrial waters may be in ferric or ferrous condition, soluble, colloidal, or insoluble. Ferric iron is rarely found in solution in appreciable quantities except in acid waters. Insoluble or colloidal iron is likely to be all ferric.[1]

An iron content greater than about 0.2 p.p.m. is undesirable in a water supply; when it exceeds 0.5 p.p.m., the taste is noticeable and disagreeable. The U.S. Public Health Service Standard limit is 0.3 p.p.m. Its concentration in potable waters is insufficient to have any hygienic significance. Sulphate iron causes acidity and corrosive action on iron and brass. Bronze or lead-lined pipes are required for carrying it.

Dissolved compounds of iron can be removed from water by sedimentation or filtration, or both, by filtration alone, by the removal of carbon dioxide, by zeolite action, and by the other methods.

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[1] "Standard Methods for the Analysis of Water and Sewage," 8th ed., p. 73, American Public Health Association, 1936.

Aeration, either with or without filtration, is especially applicable when iron is present as bicarbonate alone. It is probably the method of iron removal most extensively used. The amount of oxygen required for the oxidation of ferrous iron is only about 0.14 part per part of iron, so that slight agitation with air is usually sufficient. The oxidation of ferrous iron may be accelerated, particularly in waters high in carbon dioxide, by raising the pH through aeration or the addition of such alkaline substances as lime, soda ash, or caustic soda. Acid waters must be made alkaline. Aeration can be overdone, due to excessive agitation, and thus increase the difficulty of iron removal. Sufficient aeration is frequently obtained through the ordinary handling of water in open sedimentation basins preceding filtration. The precipitated iron may be removed by allowing the water to trickle over contact beds of coke or similar material. The iron removal is enhanced by catalytic action of previously deposited hydrates or oxides of iron. Settling basins, following aeration, may be designed for retention periods of 30 min. to 2 hr. and should be equipped for ease in cleaning.

The presence of organic matter, manganese, and carbon or organic acids renders the removal of iron difficult. It may be necessary to add lime or to allow the aerated water to trickle over contact beds of broken stone or coke. Filtration alone is effective due to biologic action dependent on a growth of organisms on the filter sand. Carbon dioxide may hold certain forms of iron in solution. Its removal will result in the precipitation of the iron, which can be removed by sedimentation or filtration.

The zeolite process of iron and manganese removal involves oxidation and filtration, not base exchange. It is applicable mainly to clear waters that do not contain large amounts of iron or manganese. The zeolite is made by treating the zeolite used in water softening with manganese sulphate and potassium permanganate, thus causing the deposit of higher oxides of manganese. As raw water meets this prepared zeolite the iron and manganese are oxidized to insoluble hydrated oxides that are removed by the mechanical filtering action of the zeolite bed. The bed must be washed and must be regenerated occasionally with potassium permanganate. In the removal of manganese an exchange rate of 390 grains of manganese per cubic foot of zeolite, using an average of 1.78 lb. of potassium permanganate per pound of manganese removed, has been reported.[1] Iron and manganese are removed in the softening of water by zeolites, but unless the zeolite is especially prepared there is danger, if air is mixed with the water, of clogging the pores of the zeolite sand with deposited iron. Another objection is the undesirability of softening the water to zero in order to remove the iron, and the by-passing of untreated water to avoid this may leave too much iron or too much manganese in the finished product.

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[1] See also Davis, D. E., "Observations on Zeolites in Water Softening and Demanganization," *Jour. A.W.W.A.*, October, 1937, p. 1515.

The oxidation of iron, particularly of organic origin, can be effected by the use of potassium permanganate. The permanganate is most effective, and is less likely to produce a taste, if applied in the pH range from 8.8 to 9.8. In one plant chlorine[1] is used to precipitate iron from water to which lime has been added. The iron is precipitated as ferrous carbonate. In softening water by the excess-lime process, all the iron and manganese are removed, in an oxidized condition. A pH of 8.2 is needed for the removal of iron alone, or as high as 9.6 for the removal of manganese. Iron and manganese removal can be effected by chlorination beyond the break point[2] at normal *pH* values, by the application of chlorine preceding contact aeration, sedimentation, or filtration, alone or combined.

644. Manganese and Its Removal.[3] The presence of manganese in water results in the formation of brown spots on laundered goods and on paper in the process of manufacture; fermentation processes are disadvantageously affected; in dye works the colors become spotty; and a taste may be imparted to the water. It requires about 5 p.p.m. of manganese to impart taste, but only 0.5 p.p.m. may result in other undesirable characteristics. Since iron and manganese occur so frequently together, the limitations of a total of about 0.5 p.p.m. for the sum of the two metals may obviate difficulties.

Manganese compounds are found in water more frequently than is commonly appreciated. In surface waters manganese may occur in organic or colloidal form. In acid mine waters it may occur as manganous sulphate. In most ground waters it occurs as manganous hydroxide, $Mn(OH)_2$. According to Kuhr[4] it occurs also as manganous sulphate and manganous carbonate. It is frequently found mixed with iron and its compounds.

Manganese and its compounds are more difficult to remove than iron and its corresponding compounds. Methods used for the removal of manganese are similar to those used in the removal of iron. The two are frequently removed together in the same process. They may be precipitated from aerated water by the addition of lime. A pH of 9.4 is usually adequate for the precipitation of manganese. Manganous hydroxide can be oxidized to manganic oxides of the form Mn_xO_y and can be removed by filtration through sand, as at Amsterdam, Holland.[1] They are not easy to remove by sedimentation. Chemical oxidation alone will not take place in waters with a pH less than 10, but in more acid waters oxidation is effected biologically. Oxidation of manganese has been proved to be the result of the activities of *Crenothrix magnifera*.

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[1] See also HALE, F. E., "Iron Removal without Aeration," *Jour. A.W.W.A.*, October, 1936, p. 1577.

[2] MATHEWS, E. R., "Iron and Manganese Removal by Free Residual Chlorination," *Jour. A.W.W.A.*, July, 1947, p. 680.

[3] See also KUHR, C. A. H. VON W., "Manganese in Water Works," *Jour. A.W.W.A.*, Vol. 18, p. 1, 1927. and ZAPFE, CARL, "The History of Manganese in Water Supplies and Methods for Its Removal," *Jour. A.W.W.A.*, Vol. 25, p. 655, 1933.

[4] See also KUHR, *op. cit.*

Manganese has been removed successfully by chlorination, followed by sedimentation and filtration, as at Wellsburg, W. Va.,[2] and it has been removed by filtration through manganese zeolite. The removal of manganese on contact beds, followed by aeration, has been practiced successfully, particularly where pyrolusite, a native manganic oxide, is used as the contact material. Mendelsohn[3] reports its successful removal by the application of potassium permanganate in addition to coagulants, preceding filtration. Craig and others[4] removed manganese from the water supply of Providence, R.I., by the application of copperas and lime, maintaining the pH at 10 for coagulation.

645. Removal of Silica. Dissolved silica should be removed from water to be used in steam boilers to avoid the formation of hard heat-insulating scale that is difficult to remove. If aluminum is present in the water a sodium aluminum silicate scale, called analite, may be formed with very low amounts of silica. Colloidal silica can be removed by the application of metallic hydroxides or oxides to the water. Ferric sulphate, when added to water containing sufficient calcium hydroxide, will produce ferric hydroxide, useful as a silica absorbent but involving an undesirable increase in total dissolved solids. Other silica absorbents now in more general use include insoluble magnesium oxide or magnesium hydroxide or the precipitation of ionic magnesium from solution. Dissolved silica is more difficult to remove, and much study is being given to methods for removing it.

646. Removal of Fluorides. Many methods for the removal of fluorides from water have been proposed,[5] but none has been extensively tried in practice. Scott[1] has proposed an overtreatment with lime to a causticity of about 2.0 grains per gal. in the presence of sufficient magnesium. The relation between the residual fluorides Y and the magnesium removed X is expressed by Scott as

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[1] Kuhr, *loc. cit.*

[2] See also Davis, M. J., "Demanganization by Prechlorination," *8th Ann. W. Va. Conference*, p. 44, 1933; and *Water Works Eng.*, Nov. 29, 1933, p. 1192.

[3] Mendelsohn, I W., "Water Supply and Purification Developments during 1936," *Water Works & Sewerage*, January, 1935, p. 3.

[4] Craig, E. C., and others, "Iron and Lime in Removal of Manganese," *Jour. A.W.W.A.*, Vol. 24, p. 1762, 1932.

[5] See also Smith, H. V., and M. C. Smith, "Bone Contact Removes Fluorine," *Water Works Eng.*, Nov. 10, 1937, p. 1600.

$$Y = F - 0.07F\sqrt{X} \tag{1}$$

in which F is the original fluorine content. This procedure can be followed, with practically no modifications, in standard lime-soda softening plants. The method is costly, however, and requires competent chemical control. The Smiths[2] proposed a method involving contact of the water with specially prepared ground bone. In their tests, 2 g. of bone reduced the fluorine in 500 cc. of water from 6 to 0.9 p.p.m., a concentration lower than that necessary to cause mottling of human teeth. The method is simple, relatively inexpensive, and has the advantage of being applicable to household as well as municipal installations without expert control. Fink and Lindsay[3] have conducted successful experiments in the reduction of fluorides to less than 1.0 p.p.m. by passing the water through a bed of activated alumina. This process has not been applied to a public water supply, but equipment is available for the treatment of small domestic and private water supplies.

647. Removal of Certain Dissolved Metals. Traces of lead, zinc, copper, and tin can be removed from water by the use of zeolite,[4] in much the same manner as zeolites are used in water softening. The zeolite used in successful tests for the removal of these substances were known as Kenzelite and Zepholite. Each was regenerated with sodium chloride, except in the case of lead, which required regeneration with sodium nitrate.

648. Removal of Color.[5] Color is caused by colloidal matter in water. It may be removed, as other colloids are removed, by adsorption and by chemical precipitation or coagulation, possibly followed by sand filtration. No one method will remove all colors. Other methods include bleaching by sunlight in open reservoirs or by the use of chlorine. Coagulation with alum, followed by filtration, is probably the most widely used procedure. About 1 grain of alum per gallon of water should remove about 10 p.p.m. of color under favorable conditions. In the removal of color by chemical coagulants the same principles apply as in the addition of coagulants for other purposes.

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[1] SCOTT, R. D., and others, "Iron and Lime in Removal of Manganese," *Jour A.W.W.A.*, January, 1937, p. 9.

[2] SMITH and SMITH, *op cit*

[3] FINK, G. J., and F. K. LINDSAY, "Activated Alumina for Removing Fluorides from Drinking Water," *Ind. Eng. Chem.*, August, 1936, p. 947.

[4] See also ADAMS, B. A., "Removal of Lead, Zinc, Copper, and Tin from Drinking Water by Means of Base-exchange Materials," *Water and Water Eng.*, September 20, 1930, p. 415.

[5] See also HOWARD, N. J., "Treatment of Colored and Corrosive Waters," *Jour. A.W.W.A.*, December, 1937, p. 1916.

649. Removal of Taste and Odor.[1] Conditions creating tastes and odors in water determine, to a great extent, the method employed in their removal. The causes of tastes and odors may be divided somewhat as follows: (1) organic and vegetable matter, (2) industrial wastes and domestic sewage, (3) dissolved gases, and (4) dissolved mineral matter. Since tastes and odors are closely associated the removal of one will, generally, remove the other.

Methods for the control of tastes and odors include any one or a combination of the following: (1) aeration, (2) prechlorination, (3) superchlorination followed by dechlorination, (4) chloramine treatment, (5) chlorine dioxide, (6) activated carbon, (7) potassium permanganater,[2] (8) copper sulphate, (9) bleaching clay,[3] (10) lime alone or combined with ferrous hydroxide, (11) ozone, and (12) the removal of iron and manganese. None of these methods is applicable to the removal of all tastes and odors, nor are all widely used. For example, tastes and odors due to the presence of inorganic compounds may be controlled best by aeration or the several methods of chemical treatment. Aeration or activated carbon is especially applicable to the removal of tastes resulting from dissolved gases, whereas chloramine or lime[4] is applicable in the removal of phenol tastes. Where tastes have been exceptionally severe the combination of prechlorination and split treatment with activated carbon added also just ahead of the filters may be effective. Potassium permanganate has been found effective in controlling some tastes produced by microorganisms; Gibbons[5] found that it removed tastes and odors caused by solvent naphtha wastes, rosin, soap, and fermenting cereal wastes. It may be used alone or in combination with chlorine. The dose and application should be controlled so that no pink color passes the filter. Doses between 0.2 and 0.4 p.p.m. have been reported.

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[1] See also Bailey, W. T., "Taste and Odor Control at Council Bluffs," *Jour. A.W.W.A.*, April, 1935, p. 458; Baylis, J. R., "Elimination of Taste and Odor in Water," McGraw-Hill Book Company, Inc., New York, 1935; Gullans, Oscar, *Jour. A.W.W.A.*, July, 1933, p. 974; and Baylis, J. R., and O. Gullans, *Jour. A.W.W.A.*, April, 1936, p. 507.

[2] See also Houston, Alexander, *Munic. Sanit.*, April, 1932, p. 148; and LaShell, L. L., *5th Ann. Rept. W. Va. Conference on Water Purification*, 1930, p. 37; and Haynes, L., *ibid.*, p. 35.

[3] See also Dodd, R. I., "The Use of Absorbing Clay for Removing Taste and Odor Producing Substances from Water," *Jour. Penna. Water Works Operators*, 1932, p. 65.

[4] See also Scott, J. S., "Control of Phenol Tastes by Means of Increased Lime Treatment," *9th Ann. Rept. Ohio Conference on Water Purification*, p. 61, 1929.

[5] See Howard, N. J., "Water Supply and Purification during 1932," *Water Works & Sewerage*, January, 1933, p. 1.

Chlorine dioxide has an oxidizing capacity $2\frac{1}{2}$ times that of chlorine. It is said to be effective against both phenolic and algal tastes, the effects being rapid. Chlorine dioxide is so unstable that it is[1] generated at the point of use by introducing sodium chlorite solution into the chlorinator discharge line. It is introduced in a proportion of equal weights of chlorine and chlorine dioxide, with concentration of 0.2 to 0.3 p.p.m. sodium chlorite. The theoretical ratio of chlorine to chlorite is 1:4, but a ratio of 1:2 or less is recommended to obtain efficient evolution of chlorine dioxide. The chemical is most economically applied at any point after oxidation of the organic matter by chlorine. It is as easy to apply as chlorine and costs no more than activated carbon.

The most widely effective treatment, where it is practical, seems to be superchlorination followed by a later application of activated carbon or the use of activated carbon alone. In a comparison of odor removal by coagulation and filtration, ozone, chloramine, superchlorination, rapid sand filtration, and aeration, Gullans[2] found that the use of activated carbon was the only practical and effective method.

650. Removal of Dissolved Gases. Dissolved gases, other than oxygen and nitrogen, can be reduced by aeration, as a result of the physical reduction of their partial pressure when atmospheric oxygen and nitrogen are dissolved. Dissolved gases can be removed by boiling, by decompression, and by the addition of chemicals. For example, either calcium or barium hydroxide will aid in the removal of carbon dioxide. The latter chemical is toxic. It is used in the treatment of boiler feed water but not for public water supplies. Carbon dioxide can be reduced also by passing the water over beds of crushed limestone.[3] Such beds may be contained in closed tanks like pressure filters, called neutralizing filters. If aeration is insufficient, the hydrogen sulphide concentration can be reduced by diffusing scrubbed flue gases through the water. Precipitated sulphur compounds should be removed by filtration to avoid reversion to hydrogen sulphide.

651. Aeration. Water is aerated to remove tastes and odors, to aid in the removal of iron and manganese, to remove undesirable dissolved gases, such as carbon dioxide, to increase the content of dissolved oxygen, and for other purposes.[1] Tastes and odors are reduced by aeration because of the removal of gases and volatile organic matter which are dissipated into the atmosphere. Atmospheric oxygen has very little oxidizing effect on organic matter in water during aeration. Natural waters that are so highly deficient in oxygen as to affect the taste are rarely found; but when they are found, aeration will restore the oxygen content and will improve the taste. Microscopic organisms may be killed not only by the oxidation, but by violent agitation given the water in the process of aeration.

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[1] See also Berry, A. E., "New Taste Control Treatment to Be Used in Lindsay, Ontario," *Water and Sewage*, February, 1945, p. 21; and Mounsey, R. J., and M. C. Hagan, "Taste and Odor Control with Chlorine Dioxide," *Jour. A.W.W.A.*, September, 1946, p. 1051.

[2] See also Gullans, Oscar, "The Comparison of Odor Elimination Treatments," *Jour. A.W.W.A.*, January, 1937, p. 60

[3] See also "Water Conditioning Handbook," Permutit Co., 1943.

Water may be aerated by (1) exposing it to the atmosphere, as in open aqueducts, basins, and reservoirs; (2) flowing over cascades, weirs, steps, troughs, etc.; (3) flowing through trickling devices such as coke beds and perforated trays; (4) spraying it through the air; (5) diffusing air through it by means of perforated pipes, porous plates, strainers;[2] (6) aspirating air through the water,[3] mixing air and water under pressure to increase the solubility of oxygen; and other methods.[4] The first four methods involving the exposure of water to air are most commonly used. Many proprietary aerators of different types have been produced. Beds about 9 in. deep of 2- to 3-in. coke held in perforated trays, with three or four trays, one about 4 to 6 in. beneath the other, are widely used. If it is desired to remove the entrained air bubbles after aeration, the water may flow from the aerator into a basin with a retention period of about 15 min. to 2 hr. and adequate surface area. For cascades and splash trays, water falls from 3 to 10 ft. Spray nozzles need a pressure of about 10 p.s.i. at the throat. Air-diffuser basins may have a retention period of about 15 min., a depth of 9 to 15 ft., and use 0.05 to 0.10 cu. ft. of free air per gallon of water treated, diffusing the air through perforated pipes or porous plates at the bottom of the tank, and requiring about 1 kw. of power per million gallons daily capacity.

Efficiencies of aerators are expressed in percentage removal of carbon dioxide; *e.g.*, the perforated-pipe method of aeration requiring the forcing of air through water surrounding the pipe is given a rating as high as 75 per cent.

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[1] See also HALE, F. E., "Present Status of Aeration," *Jour. A.W.W.A.*, 1932, p. 1401; and FLENTJE, M., *Jour. A.W.W.A.*, June, 1937, p. 872.

[2] See also ROE, F. C., "Air Diffusers," *Eng. and Contract Record*, Apr. 14, 1937, p. 54.

[3] See also KAUFMAN, H. L., "Determining Air Flow in Agitation Problems," *Jour. A.W.W.A.*, Vol. 25, p. 1030, 1925.

[4] See also McNAMEE, R., "'Aerators' The Purification of Michigan Waters for Public Use," *Univ. Mich. Eng. Research Bull.* 16, June, 1930.

652. Deaeration. Dissolved oxygen is removed from water principally to diminish its corrosiveness.[1] Methods for the removal of dissolved oxygen include (1) passing the water through a vessel containing a large surface of scrap iron,[2] followed by filtration through sand, all in a closed vessel under pressure; (2) the formation of ferrous hydroxide in the water by the addition of ferrous sulphate and calcium or sodium hydroxide; (3) the addition of sodium sulphite; (4) boiling the water; and (5) exposing the water to a high vacuum. In any of these methods the surface of the water must be insulated from the atmosphere in order to prevent the rapid reabsorption of oxygen from the air; in the last two methods listed the oxygen driven out of the water must be removed as fast as it is released, to prevent its reabsorption. The last two methods of deoxygenation cannot expect to remove more than about 95 per cent of the dissolved oxygen. Only chemical methods can completely deoxygenate water, but a combination of mechanical means to remove 95 per cent of the oxygen followed by a chemical treatment may be most economical. Except under careful and competent chemical control, the addition of sodium sulphite to a public water supply is undesirable because of the poisonous nature of the chemical.[3]

653. Activated Carbon. Activated carbon is a form of vegetable carbon, or charcoal, that has been heated in closed retorts in an atmosphere of such gases as carbon dioxide, air, steam, or chlorine. Occasionally chemical agents, such as phosphoric acid or zinc chloride, may be added. This "activation" and the greatly increased power of the carbon to absorb gases distinguishes activated carbon from charcoal. Most of the activated carbons used in water purification are proprietary substances sold under trade names such as Darco, Minchar, and Nuchar.

The most desired action of activated carbon in water purification is the removal of odor. Since it is not possible to measure this action by chemical analysis, tests for the quality of activated carbon aim toward a physical measurement of its odor-removing ability. Three methods have been proposed for this purpose: (1) the phenol-absorption test, (2) the dilute-iodine test, and (3) the threshold-odor test.

The phenol-absorption test[4] determines the amount of activated carbon, expressed in parts per million, required to reduce 1 p.p.m. of standard phenol in 1 l. of water to 0.01 p.p.m. after 2 hr. of stirring to assure contact between the carbon and the phenol. This test has the advantage of giving a definite analytical measurement which can be duplicated, but unfortunately it shows only the ability of the carbon to absorb phenol. Since carbons are highly selective in their action, it may give no indication of the value of the carbon for absorbing other odors. Baylis[1] suggests that the phenol absorption requirement of a satisfactory activated carbon be limited to 35 p.p.m.

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[1] See also Powell, S. T., and H. E. Bacon, "Corrosion Control by Deaeration," *Water Works & Sewerage*, April, 1937, p. 109.

[2] See Speller, F. N., "A Method for Practical Elimination of Corrosion in Hot Water Supply Pipe," *Trans. Am. Soc. Heating and Vent. Eng.*, Vol. 23, p. 125, 1917.

[3] See also Powell and Bacon, *op. cit.*

[4] See also Baylis, J. R., "The Evaluation of Activated Carbon," *Water Works & Sewerage*, June, 1933, p. 220.

In the iodine test a known weight of the carbon is shaken for a predetermined time with a solution containing a known weight of iodine. The mixture is filtered, and the residual iodine in the filtrate is determined. The iodine absorbed is a measure of the odor-absorbing power of the carbon. The test has the advantage of being simple, but it may indicate even less than the phenol test with respect to the quality of the carbon.

The threshold-odor test[2] is conducted in the manner described in Sec. 465 for the determination of the threshold-odor number, except that the amount of activated carbon required to reduce the threshold odor from one to another predetermined value is measured. It is a difficult test to make, but it has the advantage that it may measure directly the characteristic most desired in the activated carbon under observation.

In general, it can be expected that good activated carbon will weigh 10 to 11 lb. per cu. ft., that it will contain not less than 90 per cent of pure carbon, and that not over 6 per cent will be retained on a 200-mesh sieve. Activated carbon is supplied in bags weighing about 75 lb. When supplied in carload lots of 15 tons, it can be bought for about 5 cents per lb. The cost of the treatment ranges between 40 cents and $1 per million gallons.

654. Use of Activated Carbon. Activated carbon is used in water purification principally for the control of tastes and odors resulting from the presence of dissolved gases. It has been found effective in aiding coagulation in the removal of color, in preventing or retarding the decomposition of sludge in settling basins, and in the reduction of the chlorine demand of treated water. Among its advantages may be included the fact that no chemical reagent is added to the purified water, because the carbon is filtered out, there is no danger from an overdose of activated carbon, and excess chlorination is made easier.[3] If powdered activated carbon is added to the influent to a rapid sand filter in an amount less than

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[1] "Specifications for Activated Carbon," *Water Works & Sewerage*, September, 1932, p. 311; and March, 1933, p. 87.

[2] See also BAYLIS, J. R., "The Evaluation of Activated Carbon," *Water Works & Sewerage*, June, 1933, p. 220.

[3] *Jour. A.W.W.A.*, 1934, p. 6; and SIGWORTH, E. A., "Activated Carbon—Its Value and Proper Points of Application," *Jour. A.W.W.A.*, Vol. 29, p. 688, 1937.

5 p.p.m., the filter runs are not shortened,[1] but in some cases the reduction of length of run has been as much as 50 per cent.

The limits of adsorption of which activated carbon is capable has been formulated by Freundlich[2] as follows:

$$x = MKC^{1/n}$$

where x = total amount of impurity adsorbed from a fixed quantity of solution
M = amount of activated carbon used
C = residual concentration of impurity in solution, after adsorption is complete
K and n = constants

In the removal of tastes and odors from water by activated carbon the material may be applied as a dry powder directly to the water at one or more points preceding filtration or sedimentation, or filtered water may be passed through a bed of activated carbon.

If a dry powder is used it may be applied to the water before filtration, either alone or mixed with other chemicals such as alum, by means of any of the types of dry chemical feed machines. It may be fed before, during, or after coagulation or at more than one point. This is known as "split treatment." The most common point of application is preceding coagulation. Wherever the carbon is applied, it should be so controlled that a small portion reaches the filter. It may be desirable occasionally to "bump" the filter shortly after washing or after commencing the application of carbon to work it into the interstices of the sand. Dosages vary widely but average, in practice, about 20 to 25 lb. per million gallons.

The following points should be considered in the feeding of activated carbon: (1) The carbon should be handled in a separate room to minimize the dust nuisance. (2) The feeding machine should have considerable overcapacity because of the wide fluctuations of load that come to most plants. (3) A plentiful water supply should be provided to carry the carbon to the point of application without danger of clogging the solution pipe.[3] (4) The strength of solution, where solution feed is used, should be about 4 per cent. Rubber or iron pipes may be used for conveying the solution.

[1] See also GIBBONS, M. M., "Tests on Activated Carbon Show Its Selective Nature," *Eng. News-Record*, Mar. 17, 1932, p. 391.

[2] See discussion in "Water Quality and Treatment," Manual American Water Works Association, 1940, p. 160.

[3] See also HARRIS, J. P., and E. A. SIGWORTH, "The Feeding of Activated Carbon," *Jour. A.W.W.A.*, April, 1937, p. 504.

Care must be observed in the storage of these materials, as they are furnished in such a finely divided state that an explosive mixture with the atmosphere may result. Because of the high absorbtive properties of the material, enough heat might be generated in an improperly designed storage room to reach the ignition point. Storage space of 40 to 150 cu. ft. per ton should be allowed.

655. Magnetite Sand Filters. Filters of magnetite sand, made from magnetic iron ore, are in use for the treatment of either water or sewage, for the purpose of removing finely divided suspended matter by mechanical straining without the preliminary addition of coagulating chemicals. Insufficient experience has been had with the medium in water treatment[1] to establish a standard rate of filtration, but, in general, it can be assumed that a rate of 2.0 to 4.0 g.p.m. per sq. ft. for an upward-flow filter will give satisfactory results.

The filters are constructed by placing a layer, about 3 in. thick, of magnetite sand with an effective size of about 0.6 mm. and a uniformity coefficient of about 1.1 on a phosphor-bronze screen. Water to be treated passes either upward or downward through the sand and the screen. When the head loss has reached the limiting amount of about $6\frac{1}{2}$ in., the sand is cleaned by passing an electromagnet and a pump over the bed. The sand, which is magnetic, is picked up by the electromagnet, cleaned as it is pulled through the water, and washed by the water rising to the pump. The dirty wash water is discharged from the sand cleaner by the pump. The current to the electromagnet is alternately turned on and off as the cleaner moves slowly along the filter. When the current is released, the clean sand falls back into place on the screen. An advantage of the apparatus is that the entire process can be made automatic so that when the bed needs cleaning the washing apparatus is automatically put into operation.

656. Puech-Chabal and Drifting-sand Filters. These two types of filter are used for the improvement of the sanitary quality of a water, for the same purposes as slow and rapid sand filters. The Puech-Chabal filters are confined almost exclusively to France; the drifting-sand filter at Toronto, Ont., the only one of its kind in existence, has been abandoned.

In the Puech-Chabal system of filtration the water is passed at a progressively slower rate through a series of sand filters. A coagulant is not commonly used, and the water is aerated between all stages of filtration.

The outstanding feature of the drifting-sand filter is its ability to operate without interruption for sand washing, since the cleaning of the sand proceeds as the filter operates.[1]

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[1] See also ZACK, S. I., "The Magnetite Filter in Sewage and Water Treatment," *Water Works & Sewerage*, June, 1937, p. 201.

657. Diatomite Filters. Diatomite filters for the purification of water were developed, to a considerable extent, during the Second World War, mainly for the use of troops in the field.[2] Because of their mechanical limitations it is improbable that they will be used extensively for public water supplies; although they may be found suitable for special purposes in conditioning water for industrial supplies. A section through a 50 g.p.m. diatomite filter is illustrated in Fig. 257. The water to be treated enters at the bottom of the apparatus and flows upward. When the filter is restored to operation after backwashing, the necessary amount of diatomaceous earth is mixed with the entering water. This water, passing through the porous septum, leaves the diatomaceous earth or "filter aid" on the surface of the septum to act as a filtering medium. The thickness of this layer may be $^1/_{16}$ to $^1/_4$ in. As water to be filtered passes through this filtering layer the head loss increases until it has

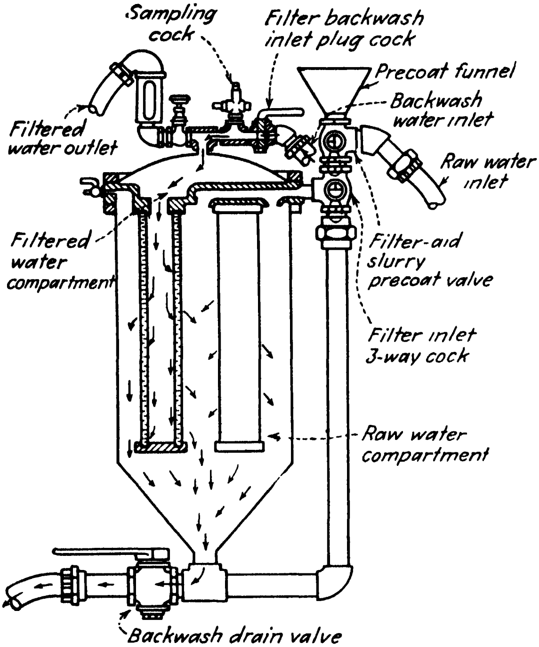

FIG. 257. Diatomite filter being backwashed.

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[1] See also NASMITH, G. G., "Toronto Test of Drifting Sand Filter," *Can. Eng.* Apr. 8, 1915.

[2] See also BLACK, H. H., and C. H. SPAULDING, "Diatomite Filtration for Field Troops," *Jour. A.W.W.A.*, November, 1944, p. 1208.

reached about 5 ft. or more of water head. The flow through the filter is then reversed and the backwash water with dirty filter aid is wasted. When the wasted water becomes clear, the washing is stopped and the normal direction of flow through the filter is resumed, a new layer of diatomaceous earth is placed on the septum, and the cycle of operation of the filter has been completed. The rate of filtration is greatly affected by the character of the water to be treated, the character of the filter aid, and other conditions. Rates from 1 to 15 g.p.m. per sq. ft. of filter surface are reported, with lower rates giving the better quality of water. From 800 to 1,000 lb. of filter aid may be used per million gallons of water treated.

Among the advantages of the use of diatomite filters for the production of potable water is the relatively low weight and volume of the filter compared with the conventional rapid sand filter. The filter will remove cysts of *E. histolytica* and probably other microorganisms not seriously affected by chlorination.

658. Chemical Control of Metallic Corrosion. The corrosion of metals as a result of electrochemical action is discussed in Sec. 334. It follows from this explanation that the corrosivity of water may be reduced by (1) controlling the carbonate balance of the water so that it will lay down a carbonate film to protect the metal from the electrolyte; (2) increasing the pH of the water, *i.e.*, by decreasing the hydrogen ions available to replace metallic ions discharged from the metal; and (3) deaerating the water so that oxygen is not available to react with the protective hydrogen film formed by electrolytic action.

659. Threshold Treatment. The application of sodium hexametaphosphate, $(NaPO_3)_6$, in a concentration of 0.5 to 1.0 p.p.m., as described in Sec. 615, has been found effective in the prevention of corrosion.[1] Close control of the dose is unnecessary, but too large concentrations may be detrimental rather than helpful. The term "threshold treatment" has been applied to this process because apparently the molecular dehydrated phosphate acts to remove the crystal nuclei from contact with the solution on the threshold of the crystallization process. The name is equally appropriate with respect to the inhibition of corrosion.[2] Precipitation of calcium carbonate is prevented by threshold treatment because the sodium hexametaphosphate segregates nuclei of calcium carbonate as they develop. The isolation or adsorption of each nucleus prevents crystalline growth.

The effectiveness of threshold treatment to prevent corrosion is in doubt. This doubt is expressed by a committee of the American Water Works Association[1] when it states that threshold treatment "will nearly always eliminate, or at least minimize the outward manifestations of corrosion" and adds that "conflicting evidence forbids a definite statement as to whether metaphosphate does or does not prevent actual corrosion of ferrous metals." Pallo[2] adds that "under conditions of low rates of flow and up to and including some critical rate of flow, the rate of corrosion was found to be higher in the threshold-treated lines than in those lines receiving no treatment."

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[1] See also RICE, OWEN, "Sodium Hexametaphosphate As an Aid in the Control of Corrosion," *Jour. Penna. Water Work's Operators Assoc.*, Vol. 12, p. 110, 1940.

[2] See also RICE, OWEN, and G. B. HATCH, "Threshold Treatment of Municipal Water Supplies," *Jour. A.W.W.A.*, Vol. 31, p. 1171, 1939.

660. Carbonate Balance. Waters containing carbonate hardness may deposit a protective carbonate film on metals exposed to the water if the carbonic acid content of the water is reduced. If the acid content is increased, the water may dissolve calcium carbonate. The deposition of calcium carbonate forms a film that protects metal from corrosion. The solution of the film exposes metal to corrosion. Too active a tendency to deposit carbonate is undesirable as it may clog pipes. To prevent clogging or corrosion by water it should neither deposit nor dissolve carbonate, *i.e.*, it should be in carbonate balance. The point of carbonate balance varies with the pH, the temperature, and other chemical characteristics of the water. This point can be determined by laboratory tests.

661. Langelier's Index of Carbonate Balance. W. F. Langelier[3] has devised an expression to indicate the pH that a water should have to be in equilibrium with calcium carbonate:

$$\text{pH}_s = (pK_2' - pK_s') + pCa + pAlk \qquad (2)$$

where pH_s = pH that the water should have to be in equilibrium with $CaCO_3$
pK_s', pK_2' = negative logarithms of the second dissociation constant for carbonic acid and activity product of $CaCO_3$, respectively
pCa = negative logarithm of molal concentration of Ca
$pAlk$ = negative logarithm of equivalent concentration of titrable base

The saturation index is obtained by subtracting pH_s from the pH of the water. A negative index indicates that the water is undersaturated with calcium carbonate. A positive index indicates the reverse. The determination of the index involves an analysis in which total solids, temperature, calcium, and alkalinity are reported. Langelier has given tables and formulas for the determination of the index, and Hoover[1] has prepared a nomograph simplifying the work of computing the index.

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[1] Committee Report, "The Value of Sodium Hexametaphosphate in Control of Difficulties Due to Corrosion in Water Systems," *Jour. A.W.W.A.*, December, 1942, pp. 1807 and 1826.

[2] Pallo, P. E., "A Study of Corrosion Control with Sodium Hexametaphosphate," *Jour. A.W.W.A.*, April, 1946, p. 499.

[3] Langelier, W. F., "The Analytical Control of Anti-corrosion Water," *Jour A.W.W.A.*, October, 1936, p. 1500.

The Langelier index is useful to determine in which direction a water is out of carbonate balance and to estimate the dosage of lime necessary to produce equilibrium. Little difficulty from corrosiveness is to be expected from cold waters with an index more positive than −0.5 and from hot waters with an index more positive than zero. It is pointed out, however, by Hoover[2] and by Ryznar,[3] that the saturation index is not always reliable because some waters with a positive index may be quite corrosive.

662. Ryznar's Stability Index. Ryznar[3] has proposed a *stability index* equal to $2pH_s - pH$ to determine the level of corrosivity or carbonate-scale-depositing potentiality of any water. The stability index will always be positive. "Without stabilizing treatment, a water having a stability index of approximately 6.0 is definitely scale forming, while an index above 7.0 may not give a protective coating of calcium carbonate scale. Corrosion would become an increasingly greater problem as the stability index increased above 7.5 or 8.0." He states also[4] that stabilization of the water with a polyphosphate, such as $Na_9P_7O_{22}$, will prevent scale formation by a water with a stability index as low as 4.0, with temperatures up to 200°F.

663. McLaughlin's Method. P. L. McLaughlin[5] has proposed a laboratory procedure to indicate the point of carbonate balance and, hence, the corrosiveness of the water. The procedure is based in the assumption that the degree of carbonate saturation, within the limits required for the performance of the test, of a water at a given alkalinity and pH is a function of the degree of increase or decrease in alkalinity of the water after treatment with an excess of chemically pure $CaCO_3$. Hence, to determine the pH of the carbonate saturation point of an undersaturated water, a curve is drawn with pH as abscissas and change of alkalinity resulting from the addition of $CaCO_3$ as ordinates, as shown in Fig. 258. In following the procedure in the laboratory any particular pH of the water is obtained by the addition of lime. At each value of pH the alkalinity is determined before and after the addition of $CaCO_3$. The change in alkalinity is plotted as the ordinate for the corresponding pH. The point of intersection of the curve thus drawn and the base line for alkalinity represent the pH of the carbonate saturation point.

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[1] Hoover, C. P., "Practical Application of the Langelier Method," *Jour. A.W.W.A.*, November, 1938, p. 1802.

[2] Hoover, C. P., "Stabilization of Lime-softened Water," *Jour. A.W.W.A.*, Vol. 34, p. 1425, 1942.

[3] Ryznar, J. W., "A New Index for Determining Amount of Calcium Carbonate Scale Formed by a Water," *Jour. A.W.W.A.*, April, 1944, p. 472.

[4] *Ibid.*, p. 478.

[5] McLaughlin, P. L., "A Procedure for Determining the Necessary Treatment to Prevent Corrosion," *Proc. 10th Ann. Conf. on Water Purification, W. Va. State Univ. Tech. Bull.* 8, p. 19, Apr. 15, 1936.

664. Chemical Adjustment of pH. Lime is the chemical most commonly used in the adjustment of pH to control corrosion either to obtain carbonate balance or to reduce the concentration of hydrogen ions available to replace metallic ions. Other alkaline compounds, such as sodium or potassium hydroxides, will have no effect on the hardness of the water, but their cost is so high, compared with the cost of lime, that their use for public water supplies is not economical.

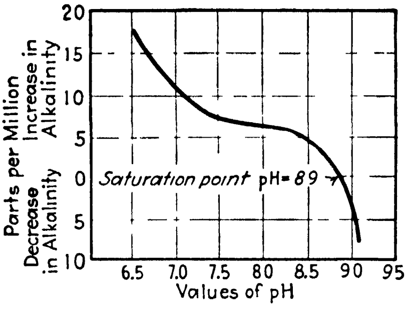

FIG. 258 McLaughlin's graph for determining carbonate balance.

665. Chemical Formation of Carbonate Coatings. Protective coatings may be formed on metals by the adjustment of the pH to lay down a coating of calcium carbonate, by the addition of sodium silicate, and by reaction between constituents of the water and of the metal.

The addition of sodium silicate to water, particularly in the presence of calcium and magnesium salts, will result in the formation of a protective silicate coating. In order to form the first protective coating, enough silicate should be added to increase the hydroxide alkalinity 15 to 30 p.p.m. Thereafter the maintenance of 5 to 10 p.p.m. will probably be sufficient to provide protection in either hot- or cold-water pipes. The use of sodium silicate in public water supplies must be controlled with intelligence, as an excessive amount, increasing the alkalinity to 100 p.p.m. or more, produces a taste, discolors foods in cooking, and produces an unctuous feeling on the hands.

The formation of protective coatings by reaction between constituents of the water and the metal is exemplified by the deposition of a protective layer of hydroxide of iron, formed by the reaction between the iron and dissolved oxygen. Similarly, lead reacts with certain hard waters to form basic lead carbonate, which deposits on the lead pipe to delay further corrosion. Such naturally formed coatings are important in the protection of many pipe lines today, and in some cases disastrous results have followed treatment of water that has dissolved these coatings.

666. Chlorination to Control Corrosion. The corrosive action of certain bacteria is discussed in Sec. 339. The addition of chlorine may aggravate rather than diminish such bacterial activity. When chlorine is added to water containing organic matter, it reacts with the matter to diminish the bactericidal effect and possibly to furnish food for aftergrowths of bacteria. The addition of ammonia with the chlorine will retard the exhaustion of the chlorine so that it may be carried into remote parts of the distribution system where its bactericidal effects will be felt further and endure longer.

In a few instances the use of ammonia has been found to aggravate the difficulties from corrosion in certain parts of the distribution system, probably because of the fact that when the chlorine is finally exhausted the remaining ammonia furnishes food for the undesirable aftergrowths Under such circumstances the remedy is to stop the application of the ammonia, to change the conditions of its application, to study the effectiveness of copper sulphate, or to attempt other methods for the control of corrosion.

667. Corrosion by Hot Water. Responsibility of the management of the public waterworks extends to the corrosion of the consumer's plumbing as well as to the corrosion of the pipes belonging to the water department. Since it is common practice on the part of consumers to heat a part of the water used and to distribute it in the plumbing system, the corrosive characteristics of the public water supply when heated should be minimized. Hot water is more corrosive than cold; corrosive characteristics of water when heated are greatly changed; and methods for correction of the corrosiveness of hot water differ somewhat from the correction of cold water. Water with a pH of 7.5 to 8.0 may be only mildly corrosive to steel at temperatures of 50 to 60°F. but will become actively corrosive at temperatures of 120 to 150°F. The complete deaeration of waters to be heated to high temperatures will greatly reduce their corrosiveness. However, the disconcerting conclusion was reached by Hoover[1] that some waters high in pH and devoid of dissolved oxygen may be highly corrosive when heated. In general, it is desirable to maintain a more positive Langelier index, or lower stability index, in hot than in cold water because of the higher solubility of carbonate in hot water.

668. Effect of pH Correction on Zinc and Copper. It is generally accepted that the higher the pH of water, the less corrosive it will be to iron, but this is not altogether true with respect to copper and zinc. In a series of tests by Moore[1] it was shown that loss of zinc from brass diminishes as the pH is raised from 6.0 to 10.0 and that up to a pH of 11.0 it increases again, that the rate of corrosion of copper from brass is inversely proportional to that of zinc, and that protective films will retard the loss of zinc only in a pH range of 7.5 to 9.5.

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[1] HOOVER, C. P., "Corrosion Control," *Jour. Penna. Water Works Operators' Assoc.*, Vol. 9, p. 106, 1936; *Water Works & Sewerage*, Vol 83, p. 384, 1936.

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[1] MOORE, E. W., "Corrosion of Brass in Water Subject to pH Corrosion," *Jour. New Engl. Water Works Assoc.*, 1934, p. 47.

APPENDIX I

OUTLINE OF REPORT ON WATERWORKS DESIGN

I. LETTER OF TRANSMITTAL

Address a letter of transmittal to the Mayor and City Council. The letter is to contain a brief statement of your reasons for doing the work and a brief outline of the work done.

II. GENERAL CONSIDERATIONS

1. Population, present and future.
2. Area within corporate limits, present and future.
3. Topographic features Verbal description and topographic map.
4 Transportation facilities and industrial conditions.
   Probable effect on water consumption.

III. INSTALLATION AND DEVELOPMENT OF WATERWORKS

1. Causes leading to installation.
2. Proposed method of payment.

IV. SOURCE OF SUPPLY

1. Sources of supply investigated.
2. Area of land owned by city and available for waterworks.
3. Description of wells.
4. Description of collecting reservoir.
5. Arrangement of pumping station and lot (show plan).
6 Information concerning surrounding conditions and possibilities of contamination.

V. CONSUMPTION OF WATER

1. Number of persons using water supply, and percentage of population.
2. Number of persons living along lines of mains, and percentage of total population.
3. Number of persons and percentage of total population to whom water is available for lengths of service pipe greater than 400 ft.
4. Statement as to assumptions regarding average daily consumption; maximum day's consumption; variations in consumption; hourly, daily, and seasonal; fire demand; duration of fire.

VI. PUMPING STATION AND MACHINERY

1. Location of pumping station.
2. Description of building. Sketch of interior arrangement.

3. Description of well-pumping machinery.
4. Description of service pumps.
5. Description of fire pumps.
6. Description of boilers and electrical equipment.
7. Details of auxiliary apparatus to be used.
8. Description of pipe arrangements.
9. Method of operation.

VII. Distribution System

1. General description of system, materials, etc.
2. Tabulate feet and miles of mains by sizes and indicate what percentage each size is of total length. Also indicate miles of mains per thousand population and per million gallons daily water consumption.
3. Location of mains in street, position, depth, etc.
4. Number of fire hydrants. Location. Average number of hydrants per mile of main.
5. Location of valves.
6. Tabulate valves as to number of various sizes of valves and number per mile of main of the various sizes.
7. Number of service connections now and later.

VIII. Distributing Reservoir

1. Location.
2. Operation and Control.
3. General description. Material, dimensions, capacity, nominal storage period in hours.

IX. Pressure and Fire Protection

1. Range of elevation within distribution system. State elevations considering top of well curbing as 100.
2. Normal water level at source of supply and anticipated level when pumping.
3. Ordinary and fire pressures (*a*) pumping station, (*b*) at hydrant in business district, (*c*) at extreme length hydrant in resident district, (*d*) at nozzle (with maximum length of hose) in business district, (*e*) at nozzle (with maximum length of hose) in residence district.
4. Number and size of fire streams available with above pressures.

X. Cost Data

Give an itemized account of the estimated cost of the waterworks, including *all* expenses. In arranging this account follow the same outline as the report has followed, in a style somewhat as follows:

General Items

Land (2 acres on corner..........St. and........St.)	$1,000
Building	4,700
Rights of way (private)	570
Total	$6,270

Pumping-station Equipment

Three 8-in. wells complete ready for service (not including pump head)	$0,000	
3 pump heads for above wells, etc.	0,000	
Total	$0,000	
Grand Total		$00,000

Finally, state estimated cost per inhabitant, and the probable assessment per lot benefited, and the general assessment.

XI. SUMMARY

A brief general description of the plant containing a statement of total cost, and cost per inhabitant.

APPENDIX II

PROBLEMS

CHAPTER I

1. The first cost of a centrifugal pump and motor is $1,500; the annual cost of operation and maintenance is $1,200; the life of the unit is 15 years; the annual rate of interest is 4 per cent. Determine (*a*) the capital invested, (*b*) the capitalized cost, (*c*) the annual expense, (*d*) the straight-line rate of depreciation, (*e*) the sinking-fund rate of depreciation, (*f*) the amount of the annual straight-line depreciation, (*g*) the amount of the annual sinking-fund rate of depreciation, (*h*) the value of the unit after 10 years of service, assuming sinking-fund rate of depreciation.

2. Solve the previous problem on the assumption that the salvage value of the unit is $250.

3. In the selection of a pumping unit two choices are available: unit *A* with a first cost of $5,000, an annual operating cost of $600, and a life of 15 years; and unit *B* with a first cost of $4,000, an annual operating cost of $800, and a life of 20 years. No other costs are to be considered. What will be the annual savings if the more economical unit is purchased? Rate of interest is 4 per cent.

4. The cost of a waterworks for a city of 10,000 population is $500,000. The first cost and the life of the various parts of the works are as follows: land, $10,000, life indefinite; buildings, $30,000, life 40 years; mechanical equipment, $200,000, life, 20 years; distribution system, $260,000, life 50 years. The annual cost of operation is $50,000, and the interest rate on borrowed capital is 4 per cent. The entire capital investment is covered by a bond issue of $500,000. The average amount of water sold is 80 gal. per capita per day. (*a*) If the only source of income is from water sold and the commerce commission allows 6 per cent return on capital invested, what should be the charge to the consumer for 1,000 gal. of water? (*b*) For one ton of water? Answer in cents.

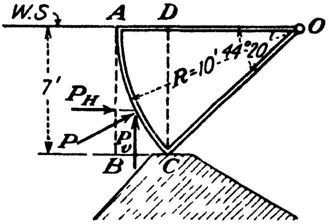
5. In developing an enterprise two choices are presented: (*a*) to install temporary equipment to function for 10 years, followed by the installation of more permanent equipment, or (*b*) to install the permanent equipment now. The temporary equipment will cost $50,000, operating cost $10,000 per year, and life 15 years. At the end of 10 years its salvage value will be $2,500. The permanent equipment will cost $100,000, operation $12,500 per year, life 30 years, and salvage value $7,500. If interest is 3 per cent, how many dollars are saved annually by adopting the more economical enterprise.

6. An amount of $8,000 is paid yearly to retire principal and pay interest on a serial bond issue of $100,000 bearing interest at 5 per cent. (*a*) How long would be required to retire the entire issue? (*b*) If all the bonds were to be retired serially in 10 years, what yearly payment would be required?

CHAPTER II

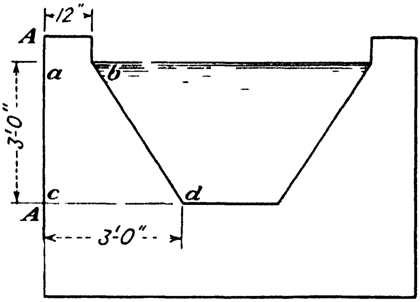
7. Calculate (*a*) the total pressure, (*b*) its line of action, and (*c*) its point of application for a circular gate leaf 6 ft. in diameter and set at an angle of 45 deg. The depth of water over the center of the gate is 20 ft.

8. The sketch shows a radial gate against which water is pressing. (*a*) Locate the distance of the center of pressure against the gate below the water surface. (*b*) What are the magnitudes of the total pressure and of its vertical and horizontal components?



9. Assume that the center (point *O* in the sketch in Prob. 8) of the gate is 3 ft. below the water surface and that point *A*, the upper edge of the gate, is at the water surface. (*a*) Locate the distance from the surface to the center of pressure. (*b*) Determine the magnitude of the total pressure and its horizontal and vertical components.

10. A trapezoidal weir, shown in the sketch, is placed in the end of a sluiceway. Locate the distance from the water surface and from the line *AA* to the center of pressure of the surface *abcd*.



11. A right triangle with a 4-ft. base and an 8-ft. side is submerged with its base horizontal and 2 ft. below the surface of the water. The plane of the triangle makes an angle of 30 deg. with the vertical. Locate the center of pressure with respect to the surface and a plane (vertical) passing through the long (8-ft.) side of the triangle.

12. A rectangular gate 20 ft. wide and 10 ft. high is set in a vertical plane. It is made to turn on horizontal hinges the centers of which are 4 ft. up from the bottom of the gate. A stop block at the bottom prevents the gate bottom from turning in the direction of the water pressure, but it is free to rotate in the opposite direction. How deep would the water flow over the top of the gate before it would open? Neglect friction and limitations imposed by pressure and velocity conditions in the stream flowing over the top.

13. A cylindrical tank 10 ft. in diameter, holding a depth of 10 ft. of water, is to be emptied through a standard circular orifice 6 in. in diameter lying in a horizontal plane in the bottom of the tank. How long, in seconds, will it take to empty the tank?

14. A conical tank is 10 ft. in diameter at the water surface and 2 ft. in diameter at the bottom. The vertical distance between the two diameters is 10 ft. How long will it take to empty the tank through a 6-in.-diameter standard circular orifice in the bottom plane of the tank? Answer in seconds.

15. A steel storage bin in a water purification plant is designed to hold a dry chemical weighing 75 lb. per cu. ft. The pressure produced by the stored chemical is assumed to act as a liquid. One side of the bin is 10 ft. wide at the top and 4 ft. wide at the bottom. It slopes at an angle of 30 deg. from the vertical so that the top of the bin is larger than the bottom. The bin is 6 ft. deep. The entire side of the bin is divided by a vertical plane into two doors with equal dimensions, the doors being hinged along the sloping edges of the bin. When the bin is filled with chemical, (*a*) where is the center of pressure on each door with respect to the top and the line where the two doors come together? (*b*) If the doors were held closed by a bolt at the top and a bolt at the bottom, what would be the maximum bending moment on each of these bolts (inch-pounds), assuming no connection between the door and the bottom of the bin? (*c*) What would be the maximum bending moment in the edge of one door in a vertical plane passing through the edge and normal to the plane of the door?

16. A railroad is investigating a new water-supply plan. The supply is to consist of a dam impounding water at *A*, from which it must be pumped over a summit at *B*. From *B* it runs by gravity to the shops at *D*. Since the head available at the shops is much more than would be used, putting in a hydroelectric plant at *C* has been suggested. The electric power will be transmitted back to *A* and used for pumping water up to *B*. The pipe line is to consist of 50,000 ft. of 24-in. cast-iron pipe and will cost about $6 per foot in place. The plant at *A* will cost as follows: dam, $75,000; land for reservoir, $5,000; buildings and machinery, $20,000. The annual operating expenses will be about $2,000. The reservoir at *B* will cost about $2,000. The plant at *C* will cost buildings and machinery $30,000; reservoir, $4,000. The annual operating expenses will be about $2,000. The transmission line between *A* and *C* will cost $20,000. The efficiencies are as follows: hydraulic turbine, 80 per cent; generator, 90 per cent; transmission line, 95 per cent; motors, 80 per cent; pump, 75 per cent. Life of masonry structure may be taken as 100 years; pipe line, 50 years; buildings, machinery, etc., 30 years. Rate of interest at $3\frac{1}{2}$ per cent.

a. What excess horsepower is there at *C*?

b. What is the capitalized and annual cost of the plant?

c. What will be the cost of water per million gallons, if 4 m.g.d. is used?

The surface of water at *A* is at elevation 1,200 ft.

There is a reservoir at *B*. Its surface is at elevation 1,400 ft.

The hydraulic turbines at *C* are at elevation 800 ft.

The length of pipe from *A* to *B* is 10,000 ft.

The length of pipe from *B* to *C* is 35,000 ft.

The length of pipe from *C* to *D* is 5,000 ft.

17. Determine the rate of discharge over a submerged weir 20 ft. wide and operating under a head of 5 ft. The submergence is 30 per cent. Contractions are suppressed.

18. A rectangular channel terminates in a sharp-crested weir whose length is equal to the width of the channel. When the velocity of flow through the channel is 47.1 f.p.m., the rate of flow is 42,500 g.p.m. When the velocity is 18.2 f.p.m., the flow is

2,000 cu. ft. per min. Determine (*a*) the width of the channel, (*b*) the length of the weir, (*c*) the maximum head on the weir, and (*d*) the minimum head on the weir.

19. An overflow spillway 30 ft. wide is so shaped that when operating under a head of 10 ft. the stream at the crest has the same cross section and the same pressure and velocity characteristics as a section at the highest point of the lower nappe of a stream discharging over a sharp-crested weir. Calculate the discharge in cubic feet per second. Contractions are suppressed.

20. Two horizontal pipes *AB* and *BC* are joined end to end. *AB* is 8 in. in diameter and 2,000 ft. long. *BC* is 4 in. in diameter and 1,500 ft. long. The pressure at *A* is 120 p.s.i., and at *C* it is 50 p.s.i. (*a*) If the coefficient *C* in Hazen and Williams formula is 100, what is the flow through the pipe in gallons per minute? (*b*) If $C = 150$, what is the flow through the pipe in gallons per minute?

21. Three pipes *ABC*, *ADC*, and *AEC* are in parallel, being joined at *A* and at *C* which are at the same elevation. Pipe *B* is 10 in. in diameter and 3,000 ft. long, *D* is 6 in. in diameter and 1,000 ft. long, and *E* is 8 in. in diameter and 2,000 ft. long. What is the pressure at *A* if the flow is from *A* toward *C* and the pressure at *C* is 20 p.s.i.? Assume $C = 100$ and $Q = 500$ g.p.m.

22. The pressure at point *A* in a pipe is 100 p.s.i. From *A* to *B* is 1,500 ft. of 8-in. pipe; between *B* and *C* there are two lines of pipe, one being 6 in. in diameter and 1,000 ft. long, and the other, in parallel with it, is 4 in in diameter and 1,500 ft. long. From *C* to *D* the pipe line consists of 1,000 ft. of 6-in. pipe. If the pressure at *D* is 80 p.s.i., and *A* and *D* are on the same elevation, what is the rate of flow through the pipe? Assume $C = 120$, and answer in gallons per minute.

23. The following values were computed from a table in Merriman's "Hydraulics" and represents the head lost *H'*, in 100 ft. of clean cast-iron pipe for various rates of flow *Q*, in second-feet. *D* = diameter of pipe:

$D = 0.75$ ft.		$D = 1.00$ ft.		$D = 1.25$ ft.		$D = 1.5$ ft.	
Q	ft./100[1]	*Q*	ft./100[1]	*Q*	ft./100[1]	*Q*	ft./100[1]
1.38	0.45	1.57	0.15	2.46	0.16	5.28	0.20
1.76	0.73	2.36	0.32	3.69	0.25	7.05	0.33
2.64	1.57	3.14	0.55	4.91	0.42	10.55	0.67
4.40	3.94	4.71	1.12	7.37	0.85	17.6	1.66
6.60	8.40	7.85	2.80	12.3	2.11	26.4	3.50
..		11.85	5.95	18.4	4.48		

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[1] This represents head lost in feet per 100 ft. of pipe.

a. Plot the values on logarithmic paper, with discharge as abscissas and lost head as ordinates, and draw a straight line through the points for each diameter of pipe.

b. Draw the lines lightly so that erasures may be made.

c. Make the lines parallel, and determine the slope.

d. For a given discharge plot *H'* against *D*.

e. Determine the slope of this line (*H'* relative to *D*).

f. Determine lines for 10-, 14-, 16-, and 24-in. pipe. (Explanation will be made by instructor.)

g. Determine *K* from relation $K = H'D^n/Q^m$, substituting one corresponding value of *H'* and *Q* for each size of pipe. Tabulate as follows:

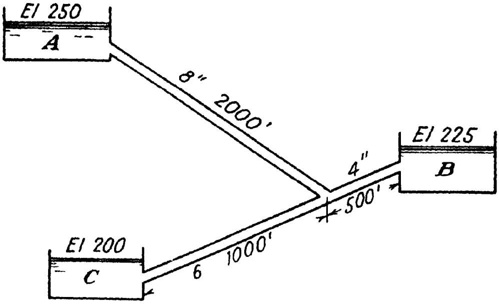
D	Q	D^n	Q^m	H'	K

h. Use the average value of *K* in equation $H' = KQ^m/D^n$

i Transpose the equation into the form $H' = K' V^{m'}/D^{n'}$

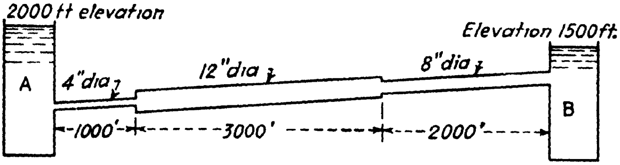
j Change equation into form $V = CS^mD^n$, and compare with Hazen and Williams formula

24. Three reservoirs are joined by cast-iron pipe, as shown in the sketch What is the rate and direction of flow in each pipe? Assume $C = 100$, and give rates of flow in gallons per minute



25. Two reservoirs are connected by 10,000 ft of riveted-steel pipe 72 in in diameter The difference in water level of the reservoirs is 16 ft What is the approximate discharge? What would the discharge be if a wood-stave pipe of the same diameter had been used?

26. Construct the hydraulic grade line and the total energy line for the pipe line shown in the sketch Use vertical scale much larger than horizontal scale Use horizontal scale 1 in = 1,000 ft Compute elevations to nearest 0.01 ft, making allowances for "minor" losses of head For friction in straight pipe, use $C = 100$



27. Two lines of hose are connected to a hydrant One line is 450 ft long and terminates at *A* in a 1-in smooth nozzle The other is 300 ft long and terminates at *B* in a 1-in smooth nozzle During a fire it is found that with *A* open and *B* closed the pressure at *A* is 45 p s i What will be the pressure and rate of discharge at each nozzle with both *A* and *B* open? Assume that pressure at the hydrant is constant under all conditions

CHAPTER III

28. Population figures for three communities are as follows:

City	1940	1930	1920	1910	1900	1890	1880
A	125,812	118,761	98,784	79,445	51,604	35,498	32,101
B	84,603	80,944	71,763	52,989	46,230	21,503	12,141
C	131,888	127,749	101,256	82,145	61,900	52,184	38,659

Predict the population of city *B* in 1960 by each of the methods described in Sec. 50.

29. (*a*) Compute the coefficient of growth for each city in the preceding problem by the method given in Sec. 51. (*b*) If each of the above cities is now within 15 per cent of "saturation," in about what decade will it be within 2 per cent of saturation?

30. (*a*) What was the maximum hourly rate of demand for high-service pumping at Toledo, in Fig. 21? Answer in gallons per minute. (*b*) What was the maximum daily high-service pumping demand during the week shown? Answer in millions of gallons. (*c*) What was the average daily rate of demand for high-service pumping for the week shown? Answer in millions of gallons.

31. Answer the questions in Prob. 30 with respect to Springfield, Mass., for the week of Mar. 17, 1940, as shown in Fig. 24. Assume that the radial scale represents rates in million gallons per day.

32. Construct a demand-duration curve for the maximum day at Toledo, similar to Fig. 25, and in accordance with the data in Fig. 22. Abscissas are to be in per cent of time for the day and ordinates in per cent of average rate for the day.

33. (*a*) Estimate the water demand by city *B*, in Prob. 28, exclusive of fire demand, for the following periods of time: (1) Maximum day, (2) average day, (3) maximum hour. Answers are to be based on the data in Tables 17 and 21 and are to be in gallons and in gallons per minute for the period covered. (*b*) What would be the proper allowance for fire protection for this city, expressed in gallons per minute and in accordance with each of the three formulas in Table 20?

CHAPTER IV

34. A 24-in. gravity well is being pumped at a rate of 300 g.p.m. Measurements in near-by test wells at this time were as follows: At a distance of 20 ft. from the well being pumped the drawdown was 20 ft., and at 50 ft. the drawdown was 6 ft. The bottom of the well is 300 ft. below the ground-water table. (*a*) If all observed points were on the Dupuit curve, what was the drawdown in the well during pumping? (*b*) Assuming the water surfaces in the adjacent test wells to have been on the "free surface," what was the drawdown of the ground-water table immediately adjacent to the well? (*c*) What is the specific capacity of the well? (*d*) What is the maximum rate at which water can be drawn from this well?

35. Assuming that the conditions in the preceding problem apply to a pressure well, answer all the preceding questions except question *b*. What is the ratio of the specific capacity of the pressure well to the specific capacity of the gravity well?

36. (*a*) Plot a curve whose abscissas are the percentage of maximum rate of discharge from the well in Prob. 34 and whose ordinates are the cooresponding percentage of total possible drawdown. Let the length of the abscissa and of the ordinate, for maximum conditions, each be 5 in. (*b*) Plot a similar curve for the pressure well in Prob. 35. Use the same abscissas and ordinates.

37. What is the specific capacity of a gravity well 12 in. in diameter and 250 ft. deep? The distance to the ground-water table under static conditions is 50 ft. When being pumped at a rate of 200 g.p.m. the drawdown in the well is 5 ft., and at 400 g.p.m. it is 12 ft.

38. A 24-in. pressure well is 450 ft. deep. The static depth of water in the well is 400 ft. When being pumped at 500 g.p.m. the drawdown is 10 ft., and at 700 g.p.m the drawdown is 15 ft. What would be the discharge from the well with a drawdown of 50 ft.?

39. To determine the velocities of flow of water through a porous-dam embankment, test holes *A* and *B* are driven into the embankment 5 ft. apart. The following observations were made: (*a*) temperature of water 62°F., (*b*) elevation of water at *A*, 589.3, (*c*) elevation of water at *B*, 584.2, (*d*) velocity of flow from *A* to *C*, 10 ft. per day. If the water in the lower part of the dam is drained down so that the elevation in hole *B* drops to 575.0, the elevation at *A* drops to 584.2, and the temperature rises to 70°F., what will be the velocity of flow of water through the dam, in feet per day? Use Hazen's formula.

40. If the distance between wells *A* and *B* in Fig. 32 is 7.5 ft., and point *A* in Fig. 33 indicates the time at which the electrolyte was added to well *A*, what was the velocity of flow between the two wells? Answer in feet per day.

41. If the well in Prob. 37 were pumped continuously at a rate of 500 g.p.m., and a borehole is sunk at a distance of 10 ft. from the well, what would be the depth at which water would be struck in the borehole during construction? (*b*) What would be the distance from the ground surface to the Dupuit curve in the borehole? (*c*) What is the drawdown in the well?

42. If it is known that the well in Prob. 37 penetrates only 50 per cent of the aquifer, what will be the discharge from the well if formula (8), page 78, is applicable?

43. Construct the drawdown curve (time as abscissas and depth of water in the well as ordinates) for the well in Prob. 37 if it is pumped at 700 g.p.m. for 5 hr. Assume $f = 0.20$. (*b*) Construct the recovery curve for the same well after 5 hr. of pumping at 700 g.p.m.

44. An artesian well was constructed 30 ft. in diameter for a depth of 40 ft. In the bottom of the 30-ft. portion a 6-in. tubular well was drilled to an additional depth of 90 ft. A test was conducted by pumping the water down in the larger diameter portion of the well and observing the rate of recovery of water level. The following observations of the distance from the ground surface to the surface of the water in the well were made: 9 A.M., 40 ft.; 3 P.M., 36.2 ft.; 9 P.M., 32.8 ft.; 3 A.M., 29.7 ft.; 9 A.M., 26.9 ft. (*a*) What quantity of water, in gallons per day, can be obtained from the well if the pump is operated to lower the water 40 ft. below the ground surface in 1 hr., daily? (*b*) What quantity of water, in gallons per day, can be obtained if the pump is operated twice daily for about ¾ hr. so as to lower the water 40 ft. below the ground surface at each period of operation? (*c*) What is the maximum quantity of water that could be pumped from the well under continuous operation with the water surface 40 ft. below the ground surface? (*d*) If the 30-ft. portion of the well were deepened by 10 ft., what quantity of water could be obtained per day from the well if the pump were operated 1 hr. daily so as to lower the water 50 ft. below the ground surface at each period of operation?

45. A number of wells, exactly like the well described in Prob. 34, are driven across a buried river valley under identical conditions. The wells are in a straight line

50 ft. apart. (*a*) What would be the combined discharge from two adjacent wells being pumped simultaneously, with no other wells being pumped, if the drawdown in each well were 20 ft.? (*b*) What would be the discharge from three adjacent wells being pumped simultaneously, with no other wells being pumped, and the drawdown in each of them were 20 ft.?

46. A waterworks owns a rectangular plot of land 300 by 519 ft. on which three artisian wells *A*, *B*, and *C* are located: *B* in the center of the plot, and *A* and *C*, respectively, in adjacent corners on the short side of the plot. Each well is 24 in. in diameter and 1,200 ft. deep. The static depth of water in each well is 1,022 ft. The log of the well showed the aquifer to be 130 ft. thick. When pumping from well *B* alone, a discharge of 490 g.p.m. was obtained with a drawdown of 98 ft. At this time the water in wells *A* and *C* was drawn down 12 ft. (*a*) What flow could be pumped from wells *A* and *C* with a drawdown in each well of 98 ft.? (*b*) What flow could be pumped from all three wells if they were pumped simultaneously with a drawdown of 98 ft.? (*c*) If two additional wells were constructed in the remaining corners of the plot, estimate the amount of water that could be drawn, in gallons per minute, by pumping all five wells simultaneously with a drawdown of 98 ft. (*d*) If each well costs $25,000, if it costs 10 cents per 1,000 gal. for power to pump water from each well, if the rate of interest plus depreciation and other annual overhead charges is 10 per cent, and if water can be sold for 15 cents per 1,000 gal., how many wells of the same type, all being pumped continuously, can be economically located on this plot, assuming the maximum possible drawdown to be 98 ft.?

47. (*a*) Tabulate the maximum rate of rainfall, the month of its occurrence, and the percentage deviation from the annual mean, in each of the 11 typical regions included in Fig. 59. (*b*) What would be the mean annual rainfall for the "Eastern Rocky Mountain" type, if the rainfall in each of the months from April to September, inclusive, were increased by 50 per cent, the rainfall for other months remaining as shown in Fig. 59?

48. The drainage area of the Scioto River, above the storage dam at Columbus, Ohio, is 1,080 sq. miles. The extreme length of the watershed above the dam is 80 miles. For the 24 hr. ending at 9 P.M. on Mar. 23, 1913, the average rainfall over the drainage area was 4.9 in. The total rainfall on the drainage area for the 48-hr. period ending at 8 P.M. Mar. 24 was 6.4 in., and for the 72-hr. period ending Mar. 25 at 8 P.M. it was 7.9 in. The storm began about 8 P.M., Mar. 22. At noon on Mar. 25 the apex of the flood was reached, with a depth on the spillway of the dam of 13.6 ft. The crest of the dam is 500 ft. long. Determine the percentage runoff, calculating the discharge over the dam from the expression

$$Q = 3.7LH^{3/2}$$

49. On a drainage area of 250 sq. miles it has been found that 22 hr. is required for the concentration of flood flow. The area, consisting of rolling cultivated land, is in Illinois. Determine the maximum probable flood flow by (*a*) the rational method, (*b*) empirical formulas.

50. From the flow data obtained from U.S. Geological Survey *Water Supply Papers* and from corresponding rainfall data, plot a hydrograph, a unit hydrograph, a flow-duration curve, and a mass curve of total flow for the period 19__ to 19__ for the ________ River at ____________. (The instructor will supply references in which necessary data can be found.)

51. Tabulate the maximum hourly rates of rainfall that may be expected once in

10 years for 5-min. storms in at least five of the region shown in Table 36. In each case use the smaller value of K, x, and n shown in the table for the chosen time period.

52. A rectangular drainage area is 10 miles long from east to west and is 6 miles wide from north to south. Rainfall observations during a storm were as folllws: at the northwest corner, 5 in.; at the northeast corner, 6 in.; at the southeast corner, 4 in.; at the southwest corner, 3 in.; and in the center of the area, 7 in. What was the average rainfall on the drainage area?

CHAPTER VII

53. Draw, to scale, the cross section of a rolled-fill earth dam holding a depth of 60 ft. of water. The dam is to be on a slightly porous clay foundation. Three classes of material are to be used: grade *A* is a highly impervious mixture of sand and clay, suitable for puddle core wall; grade *B* is only slightly pervious sandy clay; and grade *C* is quite pervious. The upstream side of the dam is to be paved, and one or more berms are to be included on each side of the dam. The extreme length of exposed water surface is 2 miles. All other conditions affecting the design are to be assumed and explained.

54. What should be the thickness of the lower edge of the apron of the dam shown in Fig. 64 to withstand uplift, assuming a factor of safety against uplift of 2?

55. Sketch a cross section of a masonry dam similar to Fig. 69, and demonstrate that the resultant of all overturning forces must pass within the middle third of the base of the dam in order to avoid tension on the upstream face of the dam.

56. Sketch a cross section of a masonry dam, similar to Fig. 69, and show the approximate location and direction of action of the resultant of each force listed in Sec. 154.

57. If the base of the dam shown in Fig. 64 is 70 ft. thick, what is the total uplift, per foot of length of dam? How far from the upstream edge of the dam will the resultant of the uplift act? Assume the distribution of uplift pressures to be linear across the foundation.

58. (*a*) What is the weight, per foot length of dam, of the dry material, grade *B*, on the upstream side of the Dear Creek Dam shown in Fig. 65? What will be its effective weight to act against overturning of the dam if the material is saturated with water and submerged beneath the water of the reservoir? (*b*) What must be the coefficient of friction of this material to prevent sliding at the upstream face?

59. What would be the thrust of the ice per foot length of a masonry dam where ice may form to a thickness of 2 ft., with no lateral restraint? Give an answer for each of the conditions included in Fig. 68.

60. If the section of the dam shown in Fig. 69 is drawn to scale and is 70 ft. high, and weighs 150 lb. per cu. ft., what must be the value of α in Eq. (7) in Sec. 160 in order that the dam will overturn when the reservoir is full? Neglect ice thrust.

CHAPTER IX

61. A trapezoidal canal with side slopes 1:1 is to be constructed to carry 1,000 c.f.s at a velocity of 4 f.p.s. What should be the base width and the depth of flow in the channel to give the greatest hydraulic radius.

62. Determine the economical size of cast-iron pipe to deliver continuously at a rate of 3 m.g.d. Pipe costs $60 per ton, hourly wages 50 cents, the cost of pumping is 5 cents per million-gallon-foot, and interest and depreciation are 7 per cent. Assume hourly wages, cost of lead, etc., are such that value of *B*, page 171, is 5.0.

63. Solve the preceding problem except that *average* delivery is to be taken as 3 m.g.d., the rate being 1.5 times the average for 12 hr. and 0.5 of the average for the remaining 12 hr.

64. Determine the economical velocity in a cast-iron pipe 24 in. in diameter if the cost of iron is $30 per ton and that of pumping is 6 cents per million gallons 1 ft. high. Rate of interest is 6 per cent. Depreciation is 1 per cent. Assume hourly wages, cost of lead, etc., are such that value of *B*, page 171, is 5.0.

65. Twenty million gallons of water per day must be pumped 1,000 ft. through a height of 30 ft. If the cost of iron, pumping, etc., are as in the preceding problem, what should be the diameter of a cast-iron pipe to convey this water?

66. A head of 85 ft. is available for an inverted siphon that has a capacity of 325 c.f.s. Neglecting losses in transitions, compute the size of concrete pipe and the size of riveted-steel pipe required.

67. Design the waterway of a bench flume to carry 500 c.f.s. if the slope is 0.006.

68. A circular aqueduct 18,000 ft. long passes through five distinctly different classes of material and will require different cross sections for economical design. The total permissible head loss is 18 ft. From the following information determine the minimum possible cost of the aqueduct and its diameter in each section.

Head loss, feet	Division 1, length 2,000 ft.		Division 2, 4,000 ft.		Division 3, 1,500 ft.		Division 4, 7,000 ft.		Division 5, 3,500 ft.	
	Area	Cost	Area	Cost	Area	Cost	Area	Cost	Area	Cost
1.0	23.0	$6,500	43.1	$7,500	20.1	$9,700				
1.5	21.0	5,700	40.1	6,500	18.9	9,200				
2.0	19.7	4,700	37.4	5,700	17.8	8,600				
2.5	17.1	3,700	35.1	5,050	16.8	8,200	20.4	$13,000	59.4	$22,000
3.0	15.9	2,500	32.8	4,400	15.9	7,800	20.0	10,800	33.8	12,800
3.5	14.8	2,200	30.9	3,900	15.1	7,400	19.6	9,300	21.0	11,000
4.0	13.7	2,300	29.1	3,400	14.4	7,000	19.3	8,000	14.6	10,100
4.5	13.0	2,400	27.3	2,800	13.8	6,600	19.0	7,000	11.4	9,600
5.0	12.5	2,800	25.7	2,500	13.3	6,300	18.7	6,300	9.8	9,400
5.5	11.7	3,300	24.2	2,200	12.9	5,950	18.5	5,500	9.0	9,100
6.0	11.2	3,900	22.7	1,800	12.6	5,800	18.3	4,900	8.6	8,950
6.5	10.6	4,500	21.3	1,650	12.4	5,700	18.2	4,500	8.4	8,800
7.0	10.1	5,300	19.9	1,500	12.2	5,700	18.15	4,150	8.3	8,700
7.5	.....	....	18.5	1,400	12.0	5,800	18.12	3,900	8.2	8,600
8.0	.	....	17.2	1,400	11.7	5,900	18.0	3,800	8.0	8,600
8.5	.	.....	15.9	1,400	.....	..	17.95	3,850	7.9	8,600
9.0	.	...	14.6	1,450	...	.	....	......	7.8	8,650
9.5	.	.	13.3	1,500	....	...	....	......	7.7	8,700
10.0	.		12.1	1,550						

CHAPTER X

69. A 24-in. steel pipe line $\frac{1}{2}$ in. thick and 300 ft. long is connected with standard cast-iron flanges. Each end of the pipe is rigidly embedded in a concrete abutment.

If the joints are 20 ft. apart and there is 20-lb. tension in each bolt when the temperature is 110°F., what will be the tension in each bolt at 32°F.? (*a*) How long must the pipe line be to stress the bolts to an intenisty of 8,000 p.s.i.? (Assume the bolts and bolt circle to be "standard.")

70. A cast-iron pipe 3 ft. in diameter is subjected to an internal pressure of 100 p.s.i. Assuming a safe working stress of 8,000 p.s.i. in tension, calculate:

a. The thickness of the shell necessary to resist the bursting pressure only.

b. If the pipe is buried in backfill which brings a load of 1,000 lb. per foot of length upon it, what must be the thickness of the shell to withstand this load?

c. In determining the final thickness of the shell, would it be safe to assume that the thickness required to resist the stress caused by the external load would also care for the bursting pressure?

71. A check valve on a 60-in. steel pipe line 3 miles long, in which the velocity of flow is 10 f.p.s., is closed in 1 sec. What is the excess pressure resulting from the closure of this valve? Assume that E for steel is 30 million p.s.i. and that the thickness of the shell is $\frac{1}{2}$ in.

72. If the pipe line in Prob. 71 is conveying water at a velocity of 4 f.p.s., and a standard 90-deg. bend with flanged ends is placed in the line, what tension would be necessary in each bolt to keep the joint tight? (For the number of bolts in a 60-in. bolt circle, see standards of the American Water Works Association.)

73. If bell-and-spigot joints were used instead of bolted flange joints, what would be the pressure, in pounds, against an abutment placed in the middle of the outside of the 90-deg. bend in Prob. 71, if the abutment were to take the full thrust due to internal pressure and the dynamic force resulting from the change in direction of the flow of water?

74. A circular concrete-lined intake tunnel from a lake intake is 10 ft. in diameter and 3.48 miles long. A quantity of 50 m.g.d. flows through the tunnel to a wet well, or suction pit, in the pumping station at the shore end of the tunnel. A vertical cylindrical concrete-lined surge tower, 6 ft. in diameter, is connected to the intake tunnel just above the entrance to the suction pit. An automatic shutoff valve is located between the surge tower and the suction pit. (*a*) What will be the elevation of the water in the surge tower during normal operation of the pumping station? Assume $C = 130$, and neglect "minor" losses of head. (*b*) What will be the magnitude, or height, of the surge wave in the surge tower if the valve is shut off in 30 sec.? Assume the velocity of the pressure wave in the tunnel and in the tower to be 3,000 f.p.s and that the coefficient of discharge of the opening into the surge tower is 0.8.

CHAPTER XI

75. The South branch of the Nashua River at the Wachuset reservoir has about 8 per cent of water surface in a total drainage area of 118 sq. miles. Monthly data of runoff and evaporation from the reservoir are shown in the accompanying table. It has been found by studies that by raising the dam sufficiently to supply a demand of 60 m.g.d. the water surface of the reservoir will be increased to 16 per cent of the watershed area. (*a*) Construct a mass diagram of runoff from the new reservoir for the period covered by the accompanying table, making proper allowance for the increased loss by evaporation, and determine, graphically, the capacity of a reservoir necessary to supply a continuous demand of 60 m.g.d. (*b*) Determine the capacities of the reservoirs necessary to supply each of the following average rates of demand, continuously, expressed in million gallons per day: 15, 30, and 90. Assume that the evaporation from each reservoir is the same as that to supply a rate of 60 m.g.d. (*c*)

What would be the time, in months, that water is flowing over the spillway of each reservoir? (*d*) What is the maximum rate of demand that could be supplied continuously from this watershed, and what is the capacity of the reservoir necessary to supply this demand, assuming the same evaporation as in part *a* of this problem? (*e*) Determine, by means of a mass diagram, the requisite capacity of an impounding reservoir capable of supplying the varying rate of demand shown in the accompanying table, assuming that the reservoir surface is 16 per cent of the watershed area. *Sugges-*

MEAN MONTHLY RUNOFF, EVAPORATION, AND DEMAND IN INCHES, ON A WATERSHED

Year	January	February	March	April	May	June	July
1908	2.57	2.68	3.75	1.96	1.90	0.62	0.34
1909	0.80	3.96	3.29	3.75	1.88	0.98	0.36
1910	2.73	2.84	2.18	1.59	0.93	1.26	0.08
Mean evaporation rate, all years	0.96	1.05	1.70	2.97	4.46	5.54	5.98
Demand, all years.	2.24	1.30	1.12	1.28	2.88	3.02	3.44

Year	August	September	October	November	December	Mean
1908	0.68	0.14	0.24	0.19	0.60	1.31
1909	0.30	0.32	0.14	0.56	0.83	1.42
1910	0.28	0.21	0.10	0.54	0.10	1.27
Mean evaporation rate, all years . .	5.50	4.12	3.16	2.25	2.51	
Demand, all years	3.30	1.96	1.54	1.22	1.51	

tion, in constructing diagrams use as a scale of abscissas 1 in. = 4 months, and for ordinates 1 in. = 12 billion gal.

76. The maximum flood flow in a stream before building a dam for an impounding reservoir was 5,000 c.f.s. lasting about 1 hr.; the flow was about 4,000 c.f.s. for 8 hr. and 3,000 c.f.s. for 16 hr. The area of the impounding reservoir is 100 acres, the length of the spillway is 200 ft., the coefficient of the spillway is 3.6. (*a*) How high will water rise above the crest of the spillway, assuming that the water was flowing 6 in. deep at the beginning of the sudden storm? (*b*) How high would water rise above the crest of the spillway to discharge 5,000 c.f.s.?

77. If the impounding reservoir for which data are shown in Fig. 93 loses, annually, 2 per cent of its present capacity due to silting, how long before it will be able to supply only one-half of the rate of demand shown in Fig. 93?

78. It may be assumed that the spillway for the Geist Reservoir, Fig. 94, is designed so that the maximum head on it will flood land to the 790-ft. contour. (*a*) If the crest of the spillway is raised 5 ft., without increasing its discharge capacity, what will be the increased storage capacity and how many additional acres of land will be flooded? (*b*) If the cost of additional land flooded is $100 per acre, and the cost of raising the crest by an overflow spillway is $25,000, how much could be saved by the construction of a siphon spillway costing $50,000 that would provide the same additional storage without flooding additional land?

79. In what length of time will the water level in the Geist reservoir (Fig. 94) rise from an elevation of 787 to 790, if the rate of discharge over the spillway is $400h^{3/2}$ expressed in cubic feet per second when h is the head of water, in feet, on the spillway and the rate of flow into the reservoir is constant at 6,000 c.f.s.?

80. Construct two routing curves, whose abscissas are time and whose ordinates are elevation of water above spillway crest, for the reservoir to which the following data are applicable:

Head above crest, ft.	0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Surface area, acres	265	270	275	280	295	300	306	312	318

The rates of inflow into the reservoir, in cubic feet per second, during each 10-min. increment of time after the flood reached the reservoir, were 1,250; 2,500; 3,750; 5,000; 6,250; 7,500; 6,875; 6,250; 5,625; 5,000; 4,375; 3,750; 3,125; 2,500; 1,875; 1,250; 625. The discharge over the spillway may be taken as $3.8\,lh^{3/2}$, in which l is the length of the spillway and h is the depth of water over the spillway, both in feet. Draw one curve for a spillway 80 ft. long and another for a spillway 200 ft. long. Then answer the following questions: (*a*) How many acres of land were saved from inundation by the construction of the 200-ft spillway? (*b*) What was the ratio of the maximum rate of overflow on the spillway to the maximum rate of flow into the reservoir?

CHAPTER XII

81. A small city purchased a centrifugal pump to be driven by an internal-combustion engine. When the bids were opened, the following proposals were found:

Name of bidder	Cost of engine and pump	Guaranteed efficiency of pump, per cent	Fuel consumption at full load, gal. per hp.-hr.
A	$1,900	70	0.12
B	1,500	65	0.10
C	1,200	62	0.11

Assuming that the pump is to deliver 900 g.p.m. against a total head of 100 ft. for 10 hr. per day and that fuel costs an average of 10 cents per gal., which is the best investment if the performances are as specified? Assume that interest plus depreciation is 15 per cent.

82. If the pumps in the preceding problem are to be driven by electric motors, with over-all costs of pump and motor the same as for engine and pump given in the table and electric power costs 1.5 cents per kilowatt-hour, what would be the cost of lifting 1 million gal. 1 ft. high if the most economical unit and type of power were selected?

83. Determine the maximum capacity of a 12-in. cast-iron suction pipe that projects 5 ft. into a reservoir. The suction lift is 20 ft; temperature of water is 60°F; and elevation above sea level is 8,000 ft. The pipe is straight, is 100 ft. long, and has a Hazen and Williams coefficient of 100. Answer in gallons per minute. NOTE: Data on vapor tensions and atmospheric pressures at various elevations must be found from other books.

CHAPTER XIII

84. A 12 by 18 by 24 by 18 by 24 outside center-packed duplex crank-and-fly-wheel pumping engine when operating at 30 r.p.m. is found to have a slip of 18 per cent. What is the rate of delivery in gallons per minute?

85. A pump and engine, with a duty of 100,000,000 based on steam, operates condensing with the use of a waterworks type of condenser. The water pressure is 70 p.s.i. at delivery. Steam is 150 p.s.i. absolute. Vacuum is 20 in. of mercury. (1) How many degrees will the temperature of the water be raised? (2) If the duty were doubled, how much would the temperature be changed? (3) If the water pressure were doubled, how much would the temperature be changed?

86. A pump with a duty of 120,000,000 (based on B.t.u.) raises 50,000,000 gal. per day of water against a head of 100 p.s.i. Neglecting steam-line and similar losses, what should be the horsepower of the boiler?

87. Illinois coal costing \$4.25 per ton will evaporate 7 lb. of water per pound of coal from and at 212°F. Which will be cheaper, and how much:

a. A 125,000,000-duty plant on a coal basis?

b. A 178,500,000-duty plant on a steam basis?

88. What is the lowest theoretical water rate possible for a simple direct-acting steam pump using dry steam, if the boiler pressure is (*a*) 20 p.s.i., absolute; (*b*) 200 p.s.i. absolute; and (*c*) 300 p.s.i. absolute?

89. What will be the power cost, in cents, of raising 1 million gal. of water 1 ft high, by each of the following pumping units: (*a*) a steam pump with a duty of 100,000,000 based on coal, (*b*) an electrically driven centrifugal pump with maximum efficiency of 75 per cent, (*c*) an oil-engine-driven centrifugal pump with over-all efficiency of 75 per cent, and (*d*) a gasoline-engine-driven centrifugal pump with 75 per cent efficiency. The cost of coal is \$4 per ton; electricity, 1.5 cents per kw.-hr.; oil, 8 cents per gal.; and gasoline, 16 cents per gal. The consumption of oil is to be taken as 0.12 gal. per hp.-hr., and the consumption of gasoline as 0.10 gal. per hp.-hr.

90. The over-all heat efficiency of a boiler in a waterworks pumping station is 80 per cent. The coal used costs \$4 per ton and contains 10,500 B.t.u. per lb A high-service steam pumping unit in the plant, discharging water at a gage pressure of 120 p.s.i , delivers 522 hp. under full load, with a mechanical efficiency of 88 per cent and a water rate of 50 lb. The boiler pressure is 150 p s i. Steam is delivered with a moisture content of 2 per cent. What is the cost of fuel used by this pumping equipment, expressed in cents per thousand gallons of water pumped?

CHAPTER XIV

91. An overhead traveling crane is to be installed in the pump room of a waterworks pumping station. It is to have a rated capacity of 10 tons. Three electric motors will be required for its operation: one to move it along the room, one to move the load sideways across the room, and one to raise and lower the load Only alternating current is available. What sizes and types of motor should you select for the services, and why?

92. A waterworks pumping station for a city of 3,000 population is to be electrically equipped for power and light. The power is to be taken from the high-tension lines near the pumping station. These lines carry current at 33,000 volts, three phase, 60 cycles, on three-wire feeders. If induction motors are to be used on the pumps, (*a*) what voltage should you use for operating them? (*b*) What voltage should you use for the lighting circuit in the building? (*c*) What electrical equipment should you install from the feeder line to each pump so that it can receive the proper current and be fully protected against any normal eventuality?

93. Make a descriptive list of the electrical equipment that should be used in a waterworks pumping station similar to that in the city of ______, except that all the electric power is to be generated at the pumping station by the use of steam. State reasons for the use of each piece of equipment.

94. A waterworks pumping station is to be electrically operated, receiving current from the power line at 11,000 volts, three phase, 60 cycles, on three-wire feeders. The station is equipped with four pumps, each capable of throwing 400 g.p.m. against a head of 120 ft., with an efficiency of 75 per cent, and two units each of 1,000 g.p.m. against a head of 240 ft., at an efficiency of 80 per cent. Make a list of the electrical equipment that you would require for the operation of the pumps, and give reasons for its selection. Indicate the provisions that you would make for safety in operation and assurance of freedom from shutdown except as a result of stoppage of current from the line.

95. If, in Prob. 94, power is to be manufactured by the municipality in its own power-and-light plant, what electrical equipment should you provide for operating the waterworks pumps? The power-generating and water-pumping equipment are all in the same building.

96. A d-c motor for low-service pumping purposes is to be installed at the bottom of a pit where conditions are wet and the atmosphere is hot and humid. The motor must deliver between 25 and 100 hp. continuously through reducing gears, to a reciprocating pump whose speed may vary between 20 and 120 r p.m. Any desired d-c voltage can be made available. The efficiency of the reducing gear is about 95 per cent. List the electrical equipment that you would install for the operation of this pump, including the equipment on the switchboard.

97. If the pump in Prob. 96 is to be used as a stand-by unit, being operated only in emergencies and starting under full load, what electrical equipment should you select for its operation, including the equipment on the switchboard?

98. An electric motor is directly connected to a centrifugal pump to lift 1,000 g.p m. of water continuously against a discharge pressure of 100 p.s.i. The unit discharges into a pipe line 2 miles long, lifting the water into an elevated tank. The pumping unit is one of many similar units in the pumping station, all discharging into the same pipe. Alternating current is available at 220 volts, 60 cycles, three phase. What size and type of motor should you use (*a*) if the unit has to start with the discharge valve open, (*b*) if the unit can be started with the discharge valve closed, (*c*) if the unit can be started so that the water can be by-passed to a low-service reservoir and the pump and motor can be operated at any desired speed and load up to the full load when the discharge valve is opened into the high-service discharge line, (*d*) if the elevated tank were shut off and the discharge pressure might lie between 60 and 120 p.s.i.

NOTE: Make necessary assumptions of data not supplied and appropriate to the conditions of the problem.

99. A waterworks pumping station has an average load of 10 m.g.d. discharged under a constant pressure of 50 p.s.i., with fluctuations in discharge as shown in Fig. 22 for the average day. The water is discharged into a distribution system on which there is elevated storage of 500,000 gal. capacity. The pumping station is equipped with three motor-driven centrifugal pumps, of equal capacity, each having an efficiency of 75 per cent at all loads. Only two of the pumping units are to be operated simultaneously, the third being held as a stand-by. Each unit can be started under no load and slowly loaded up to full load if desired. In addition the equipment is to include a Diesel-engine-driven centrifugal pump to be used as a fire pump. Rates for electrical energy delivered to the transformer at the pumping station at 2,200 volts are as follows, charges being payable monthly: first 1,000 kw.-hr. at 6 cents per kw.-hr.; next 2,000 kw.-hr. at 4 cents per kw.-hr.; next 3,000 kw.-hr. at 2 cents per kw.-hr. All over 6,000 kw.-hr. at 1.8 cents per kw.-hr. In addition there will be the following demand charges: 150 kw. or over at 0.2 cents per kw.-hr.; 250 kw. or over at 0.3 cents per kw.-hr.; and 400 kw. or over at 0.5 cents per kw.-hr. (*a*) Name the power and type of motors that you would use for each of the three units. (*b*) What would be the least cost for electric power for which you would be willing to operate this station for 30 "average" days, without using the storage on the distribution system? (*c*) State the schedule under which you would operate to provide the minimum power charge. (*d*) Solve steps *b* and *c*, utilizing the storage on the distribution system.

100. A single-stage centrifugal pump delivers 1,000 g.p.m. against a head of 250 ft. when running at 900 r.p.m. (*a*) What will be the discharge pressure and rate, at a speed of 1,200 r.p.m.? (*b*) At what speed must it be run to discharge 800 g.p.m., and what would be the head at this speed?

101. A single-stage centrifugal pump, with an impeller 10 in. in diameter, delivers 2,000 g.p.m. against a head of 100 ft. when running at 1,200 r.p.m. What is its specific speed?

102. How many stages would be required for a centrifugal pump to deliver 2,000 g.p.m. against a head of 400 ft. at a speed of 1,600 r.p.m. if the specific speed of the series is about 2,000?

103. A three-stage centrifugal pump with 12-in. diameter impellers is of the same type as in Prob. 101. At what rate and against what head will it discharge at 1,000 r.p.m.?

104. An 8-in. centrifugal pump has a straight 10-in. suction pipe lifting water 10 ft. vertically from the sump. The pump is discharging into 1,000 ft. of 8-in. cast-iron pipe (Hazen and Williams coefficient of 100) the outlet of which is 200 ft. above the pump. The temperature of the water is 70°F., and the barometer shows a pressure of 29 in. of mercury. If the efficiency of the combined pump and motor is 80 per cent, and its speed is 1,500 r.p.m., at what rate is the pump discharging, in gallons per minute, when using 50 kw. of current? Neglect only "minor" losses of head.

105. If the centrifugal pump whose characteristic curves are shown in Fig. 112 is being driven by a steam turbine with a heat efficiency of 20 per cent, what is the duty of the combined turbine and pump?

106. Compute the diameter of the impeller of the centrifugal pump whose characteristics are shown in Fig. 112.

107. What will be the least cost of power to raise 1 million gal of water 1 ft. high if electric power costs 1 cent per kw.-hr. and the centrifugal pump whose characteristics are shown in Fig. 113 is driven by an electric motor with efficiency of 92 per cent when driving the pump at its maximum efficiency?

108. Tests made on a 6-in. centrifugal pump are shown on page 582. From these data plot the characteristics, using discharge in gallons per minute as abscissas (1 in. = 200 g.p.m.) and head in feet as ordinates (1 in. = 20 ft.). Superimpose on these curves isoefficiency curves for 40, 50, 60, and 70 per cent and within 1 per cent of the maximum efficiency. Also plot horsepower curves for 5, 10, 15, 20, 25, 30, 40, 45, and 50 hp. Then answer the following questions:

a. At what speed, head, and discharge is the pump most efficient?

b. Through what ranges of head, speed, and discharge will the pump operate within 5 per cent of the maximum efficiency?

c. What is the maximum efficiency, and what is the horsepower at that efficiency?

109. What is the specific speed of the pump in Prob. 108?

DATA FOR PROBLEM 108

R.p.m.	Head, feet	Discharge, g.p.m.	Efficiency per cent	B.h.p.
1,875	121	0		
	127	200	35	
	128	400	45	
	129	450	48	
	127	730	62	
	116	1,150	74	
	100	1,500	69	
	70	1,710	56	
	50	1,730	45	
	0	1,760		
1,613	96	0		
	100	190	39	
	102	335	44	
	103	475	52	
	104	550	56	
	101.5	800	67	
	95	1,050	76	
	91	1,180	75	
	82	1,400	68	
	70	1,530	62	
	10	1,680	34	

R.p.m	Head, feet	Discharge, g.p.m.	Efficiency per cent	B.h.p.
1,425	79	0		
	83	220	40	
	86	528	55	
	84	682	63	
	80	979	75	
	78	1,045	77	
	65	1,430	67	
	42	1,672	50	
	10	1,749	33	
1,200	53	0		
	58	200	40	
	59	400	56	
	57	675	72	
	51	900	75	
	40.5	1,210	62	
	20	1,410	40	
900	35	0		
	37	250	46	
	38	395	61	
	37	550	69	
	33	750	72	
	24	1,030	62	
	15	1,240	41	

CHAPTER XVI

110. A single-stroke pump with a net valve surface of 7 sq. in. is to discharge at an average rate of 10 g.p.m. The motion of the valve is harmonic. (*a*) What is the maximum speed of travel of the valve, in feet per minute? (*b*) What is the momentary maximum rate of pump discharge, in gallons per minute?

111. A two-stroke pump, with a 30-in. stroke, each valve having a net lifting surface of 10 sq. in., operates at 40 r.p.m., the valves moving with harmonic motion. The pump is equipped with a heavy flywheel. What is the average nominal capacity of the pump, in gallons per minute?

112. The rod, or shaft, in a differential plunger pump similar to that shown in Fig. 121 is 1.5 in. in outside diameter and 80 ft. long. The weight of the rod, valve, and plunger is 420 lb. The diameter of the pipe in which the rods operate is 6 in. If the total lift from valve *C* is 40 ft., what must be the diameter of the differential plunger in order that equal work may be done by the pump on the upstroke and on the downstroke? Neglect friction. (*b*) How many gallons will be discharged on each stroke under such conditions?

113. A deep-well turbine pump is to lift 1,000 g.p.m. for a total lift of 150 ft. If the diameter of the well is to fit the pump, determine a satisfactory combination of impellers and speed within the limits fixed in Sec. 279.

114. A deep-well turbine pump is to lift 500 g.p.m. for a total lift of 300 ft. If the diameter of the impellers cannot exceed 9.5 in., how many stages must be used and at what speed must the impellers run to remain within the limits fixed in Sec. 279?

115. It is desired to raise 1 m.g.d. from an ordinary well by means of an air lift. The distance to ground water when not pumping is 100 ft. The specific capacity of wells in the vicinity has averaged about 15 g.p.m. per foot of drawdown. The water-bearing stratum consists of fine running sand. It is underlaid by impervious clay and overlaid by a shale and sand not containing water. The distance from the ground surface to the bottom of the porous stratum is 220 ft. Use smallest submergence ratio allowable within limits of lift stated on page 252.

Design the well and pumping equipment, including the following:

a. Depth of well, in feet.
b. Capacity of compressor, in cubic feet of free air per minute.
c. Size of air pipe, diameter in inches, commercial size
d. Size of eductor pipe (use same diameter pipe from top to bottom).
e. Starting and operating pressures, in pounds per square inch.
f. Velocity at bottom of eductor pipe, in feet per second.

116. (*a*) Design an air lift to raise 1,000 g.p.m. from a well, if the submergence is 60 per cent, the efficiency is 30 per cent, and the total lift is 150 ft. (*b*) If the specific capacity of the well is 10 g.p.m., what should be the starting pressure and the operating pressure of the air compressor, in pounds per square inch?

CHAPTER XVIII

117. Determine the thickness of a cast-iron pipe for the following conditions (for nomenclature and procedure see Sec. 296): d = 36 in.; S = 11,000 p.s.i.; R = 31,000 p.s.i.; water-hammer allowance = 100 p.s.i.; static pressure = 120 p.s.i.; external loads = 2,400 lb. per ft.; field condition E (Table 53) prevails. In approximately what thickness class will this pipe lie? (Table 54.)

118. First Street and Avenue A are each 40 ft. wide between curb lines. They intersect at an angle of 45 deg., First Street running east and west, and Avenue A northeast and southwest. A 24-in. bell-and-spigot cast-iron pipe line lies in Avenue A, 3 ft. northwest of the southeast curb line, running parallel thereto. The center of the spigot end of a standard 12-ft length of pit-cast iron pipe lies in the south curb line (extended) of First Street. It is desired to place a fire hydrant 2 ft. north of the north curb line on First Street and 22 ft. east of the intersection of the nearest curb lines on Avenue A and First Street. The 8-in. pit-cast bell-and-spigot iron pipe line to the hydrant from Avenue A is to run easterly on First Street, 3 ft. south of the north curb line until the right-angle turn is made to reach the hydrant. A valve is to be placed in this 8-in. line. Prepare a sketch of the layout, showing the location of pipe and specials, together with a bill of material to complete the work.

119. A pipe line 20 miles long is to be constructed to convey 25 c.f.s. of water with a loss of head not to exceed 0.1 ft. per thousand feet. (*a*) If new pit-cast iron pipe, unlined, is to be used, what would be the cost of the pipe alone? Base the size of pipe on the coefficients in Table 6 and the cost per pound at Birmingham as shown in the latest issue of the *Engineering News-Record*. (*b*) How much could be paid for centrifugally applied bitumastic lining of the best quality, without increasing the total cost for the pipe alone? (*c*) If the first costs of the entire finished job for lined and for unlined pipe are approximately equal, which kind of pipe should you select, and why?

120. In laying out the pipes in a pipe gallery of a pumping station, two flanged ends in a 12-in. line are found to be 10 ft. 3½ in. apart in a direction parallel to the axes of the two lines which are, themselves, parallel. The axes of the two lines of pipe are 3 ft. 6 in. apart in the same horizontal plane. It is desired to connect these two pipes, inserting a 12-in. gate valve in the line. Make a sketch, and prepare a bill of material for this connection.

121. A 48-in. circular aqueduct is to be constructed on the surface of the ground in a mountainous and arid region of the Southwest, where transportation is difficult and costly. The maximum internal pressure to be resisted is 15 p.s.i. Transportation charges from railhead to job are to be $5 per ton for any load of 1 ton or less. For loads above 1 ton the cost, in dollars, will be $5W^2$, where W is the weight of the load in tons. What is the minimum cost of transportation from railhead to job for each of the following materials for 1,000 ft. of aqueduct: wood-stave pipe with 3-in. redwood staves, with bands, materials for cradles, and other appurtenances weighing about 75 lb. per foot of pipe; spiral-riveted steel pipe; centrifugally cast iron pipe; transite pipe; butt-welded steel pipe; and reinforced-concrete pipe with shell 6-in. thick containing 1 part of cement to 5 of aggregate and 30 lb. of reinforcement per foot length of pipe? Material for concrete, except cement, is available at the site of the work.

122. A large Japanese city is considering the purchase of 125 miles of pipe line for an aqueduct, and the prices of the materials considered favor the use of a foreign cast-iron pipe that can be delivered f.o.b., Kobe, at 10 per cent less than American cast-iron pipe. What arguments could you present in favor of steel pipe that can be delivered to meet the price of the competing cast-iron pipe. Most of the aqueduct is to be laid across mountainous country, requiring rail transportation for 75 miles from Kobe, followed by truck transportation for a maximum distance of 15 miles to the site of the work.

123. A steel conduit 21 miles long was proportioned for pressure due to depth below the hydraulic gradient only. At station 1,052 + 96 it is 307 ft. below that gradient and 511 ft. below the reservoir surface. At this station it is 48 in. in diameter; the steel is $^3/_8$ in. thick with double-riveted lap joints in the longitudinal seams. The joints have $^5/_8$-in. rivets, staggered, on a pitch of 2.28 in. in each row. The rows are $1^1/_{16}$ in. apart. The pitch line to the edge of the plate is $^{15}/_{16}$ in., and the lap of the sheets is 3 in. Compute stress per square inch on the rivets (both shearing and bearing) and on the net plate due to the depth below the hydraulic grade line. Assume the diameter of the rivet hole to be $^1/_{16}$ in. greater than the diameter of the rivets. What is the efficiency of the joint?

124. Design a wood-stave pipe from the following data: diameter, 72 in.; maximum head, 50 ft.; allowable tension in bands, 16,000 p.s.i.; thickness of staves, $1\frac{1}{2}$ in.; swelling force of wood, 100 p.s.i.; safe crushing strength of wood, 650 p.s.i.

125. How much wire, and of what size, would be needed for 500 ft. of wood-stave pipe 3 ft. in internal diameter, subjected to a pressure head of 150 ft. of water? Size of stave 2 by 6 in., crushing strength of wood 650 p.s.i., tensile strength of wire band 20,000 p.s.i., swelling strength of wood 100 p.s.i.

126. (*a*) If the conduit described in Prob. 123 were of reinforced-concrete instead of steel, and if the cross section were the same as that shown in Fig. 160 (page 290), what should be the volume of concrete in cubic yards per foot length of conduit? (*b*) What should be the diameter of the circumferential reinforcement bars? (*c*) What should be the weight in pounds of circumferential reinforcement steel, per foot length of conduit? (*d*) What should be the diameter of longitudinal reinforcement bars? (*e*) What should be the weight of longitudinal reinforcement bars, in pounds per lineal foot of conduit?

127. (*a*) How many turns of the wheel are necessary to open the valve in Fig. 165? (*b*) Is this valve right-handed? (*c*) Determine from the scale of the figure about how many turns of the handwheel are necessary to open the valve in Fig. 169, assuming the pitch of the threads on the valve stem to be the same as in Fig. 165.

128. Determine from the scale of the figure the approximate minimum length of key necessary to open the valve shown in Fig. 168.

129. How many pounds pull would be required to start the opening of the valve shown in Fig. 170, under the following conditions: unbalanced pressure against valve disk, 50 p.s.i. acting on the disk of a 10-in. gate valve; coefficient of friction of gate against seat, 0.2; area of contact between disk and seat is an annular ring, 10-in. inside diameter and ½ in. wide; length of the valve-handle lever, 18 in.; effective distance from the valve stem to the fulcrum, 6 in.? Neglect all other friction.

130. What will be the effect on the operation of the check valve shown in Fig. 175 if valve *M* is closed?

131. If the length of the discharge pipe from the check valve shown in Fig. 174 is 300 ft., and the velocity of flow through the 12-in. pipe shown is 7 f.p.s. at the moment that flow begins to stop, what will be the total pressure, in pounds, against the valve disk when the valve closes, if the area of the disk is 118 sq. in.? (*a*) Assume the presence of no air chamber. (*b*) Assume that the air chamber is 12 in. in diameter and is two-thirds full of air. In the second case answer approximately only.

132. What will be the effect on the operation of the valve shown in Fig. 180 if valve *A* is closed?

133. If the opening of the valve shown in Fig. 186 is 12 in. in diameter and the valve is at the bottom of a reservoir with water 20 ft. deep over it, what weight must be lifted in order to open the valve?

CHAPTER XXI

134. If the average daily flow for Nov. 2, 1939, in Fig. 23, is 100,000 gal., what should be the capacity of an equalizing reservoir to equalize the rate of pumping if (*a*) pumping is to be continuous? (*b*) the pumps operate at a constant rate and operate only from 4 A.M. to 6 P.M.?

135. If the line for the average day in Fig. 22 (page 56) indicates the rate of pumpage before the installation of an equalizing reservoir, if the pressure pumped against varies as the square of the rate of pumping, if the pressure pumped against when *Q* is 1 m.g.d. is 50 p.s.i., if the population is 100,000, and if the cost of pumping is 6 cents per million gallons raised 1 ft., what will be the approximate daily saving in pumping cost as a result of installing the equalizing reservoir?

136. If the average hourly rate of pumping on an average day is 50,000 gal. per hr., how much equalizing storage capacity would be required on the maximum day if the demand were as shown in Fig. 22 (page 56) and the pumps were operated at a constant rate and for only 12 hr. per day. Solve by two methods. Use Toledo data.

137. Solve the preceding problem, using the minimum-day demand curve instead of the maximum-day curve, and assume that the pumps run at a constant rate for only 8 hr. per day.

138. (*a*) What is the capacity of the tank necessary to equalize the pumping at a continuous rate in Fig. 194? (*b*) If the distance from the ground to the bottom of the tank is 100 ft., and the tank is 20 ft. deep, plot a curve with time as abscissas and pressures in pounds per square inch in the distribution system immediately adjacent to the tank as ordinates.

139. A pipe line is 8 in. in diameter and 5,000 ft. long with $C = 120$. An elevated tank with a height of 140 ft. from the ground to the base of the tank and a depth of 20 ft. is connected to the pipe line 4,000 ft. from the pumping station, with the ground at the same elevation at the tank and at the pumping station. (*a*) When the flow from the end of the pipe line is 2,000 g.p.m., the pressure at the end is 60 p.s.i., and the tank is not connected to the pipe, what must be the pressure at the pumping station? (*b*) If the elevated tank is full and it is connected to the pumping station, what must be the pressure at the pumping station when the flow in the pipe line is 2,000 g.p.m?.

140. An earth reservoir to hold 15,000,000 gal. of water will cost \$40d^2 per foot of length of wall, where d is the depth of the water in feet, and 50 cents per sq. ft. of reservoir area. What will be the least cost for which this reservoir can be constructed?

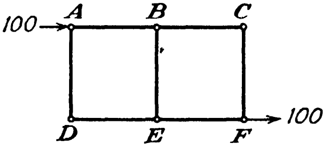
141. Determine the economical height H of a steel standpipe of capacity Q with the following unit costs and conditions: cost of steel is a cents per lb.; thickness of steel varies as bH^2 and as cD, where b and c are constants and D is the diameter of the standpipe. The cost of the foundation varies as eD^2, where e is a constant. All other costs are represented by the constant k.

CHAPTER XXII

142. A city has rectangular blocks 300 by 400 ft. The number of 250-gal. fire streams concentrated near one point is 20. Design the water-pipe system for the immediate district so that the loss of head shall not exceed 10 ft. per thousand feet. Use smallest sizes of pipe possible. Design economical layout. NOTE: Pipes will not be all the same size.

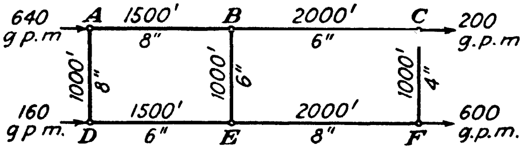
143. How many two-way hydrants throwing 200-gal. streams could be located on an 8-in. pipe 1,250 ft. long if the hydrant spacing were 500 ft. and the loss of head in the pipe should not exceed 10 lb.?

144. Compute the flow distribution in each pipe of the crossover shown in the accompanying sketch, when the relative resistances of all the pipes between letters are the same. Use the Hardy-Cross method, tabulating your computations in "standard" manner outlined in Sec. 388.



145. Water is entering a pipe junction at 1,000 g p.m. It is leaving the junction through three pipes A, B, and C whose relative resistances are, respectively, 4, 9, and 16. The pipes are joined again at the ends opposite the junction from which they drain. What is the rate of flow in each pipe in gallons per minute? Solve by the method outlined in Sec. 389, assuming that $V = kH^{\frac{1}{2}}$.

146. (*a*) Compute the rate of flow in each pipe shown in the accompanying crossover, following the method outlined in Sec. 389. (*b*) Using the preceding distribution of flow, correct it to the nearest 0.5 per cent by the Hardy-Cross method. (*c*) Construct the 5-ft. piezometric contours (lines of equal pressure) with the pressure at F equal to 75 ft. of water. Redraw the sketch to scale, and show the contours in their correct location. Use $C = 100$. (*d*) Explain the discrepancies found between the flow distribution in *a* and in *b* above.



CHAPTER XXIV

147. How many grains of hardness are equivalent to the various concentrations of hardness expressed in parts per million in Table 76.

148. If. *B. butschlii* and *Dialister pneumosintes* were shaped as cylinders, how many of each could be packed into 1 cc.? Answer in millions.

149. Name the processes of water purification shown in Table 82, which rendered potable, according to the Standards of Quality of the U.S. Public Health Service, the water supplies at Toledo, Youngstown, Niagara Falls, Cincinnati, and Louisville.

150. The Hume Reservoir on the Murray River, in Australia, was heavily impregnated with anabaena in December, 1929. At the time of the condition the reservoir was about 30 miles long and 3 miles wide, with an average depth of about 5 ft. and a maximum depth at the dam of 30 ft. Estimate the cost of applying copper sulphate, if the cost of the chemical is 1 cent per lb., and the cost of labor and equipment for applying the chemical from a power boat running at 6 m.p.h. is $2.25 per hour.

CHAPTER XXV

151. A sample of water is found to have a turbidity of 200. (*a*) How can its turbidity be changed in the laboratory to 150? (*b*) To 400?

152. Study an analysis of your home-town water supply, or some other public water supply, and write a report on its sanitary quality as compared with U.S. Public Health Service standards (Sec. 451) and with Table 79.

153. One hundred milliliters of water, saturated with oxygen at 70°F., is mixed with 200 ml. of water saturated with oxygen at 50°F. What will be the temperature and the percentage saturation with oxygen of the mixture?

154. The oxygen dissolved in the Des Plaines River at Riverside, Ill., has sometimes been found to be about 12 p.p.m. for a temperature of 30°C. (*a*) What is the percentage of saturation under this condition? (*b*) What is the explanation of the phenomenon? (*c*) Would the condition indicate a potable water, if all other conditions were satisfactory? Why?

155. Compute the amount of oxygen that is soluble in pure water at 60°F. and 760 mm. of mercury, when the surface is exposed to the air, containing 80 per cent nitrogen and 20 per cent oxygen. The result should be precise to the nearest 0.1 p.p.m. (Suggestion: Consult Whipple, *Jour. Am. Chem. Soc.*, Vol. 33, p. 364, 1911.) Is it possible to express the relationship between temperature and dissolved oxygen under the preceding conditions, mathematically? Why?

156. Prepare a hypothetical analytical report on the sanitary analysis of a water supply that you think should be condemned. The report should be prepared in a manner similar to that recommended by your state health authority. Your reasons for condemning the water should be clearly stated.

157. On the evaporation of 1 l. of aqueous solution of sodium chloride it is found to contain 42 g. of NaCl. What was the concentration of NaCl in the solution? Answer in percentage by weight and in parts per million.

158. Water enters a purification plant with a pH of 6.0. The next day the pH is 8.0. What was the average pH for the 24-hr. period, assuming linear variation of hydrogen-ion concentration.

159. Of two bases that are slightly dissociated, one has a dissociation constant of 0.000018, and the other of 0.000072. What are their relative strengths?

160. If a molal solution of a nonelectrolyte will freeze at 1.86°C. lower than pure water and a molal solution of HCl will freeze at 3.68° below 0°C., what is the percentage dissociation of the HCl? What is its pH?

161. Compute the dissociation constant of a 0.1 molal solution of zinc sulphat 45 per cent dissociated. (*a*) What is the pH of a 0.01 molal solution of acetic acid (*b*) What is the pH of a molal solution (R)OH that is 0.42 per cent dissociated?

162. What was the most probable number of *E. coli* per 100 cc. as determined b the following series of observations: of five 10-cc. portions, four were positive and one negative; of three 1-cc. portions, two were positive and one negative; and on 0.1-cc. portion was negative?

163. What was the *E. coli* index at a filter plant where the following observations were made for 2 months:

Quantities tested, cc.	1.0	0.1	0.01	0.001
Number of tests	60	40	60	30
Number positive	40	6	6	1

164. A microscopic analysis of a public water supply is shown on page 130 of "The Microscopy of Drinking Water," by Fair and Whipple, John Wiley & Sons, Inc., 1927. What are the probabilities of this water's causing complaint by the consumers? What would be the probable nature of the complaint? How should you proceed to overcome the difficulty?

165. Prepare hypothetical combinations for the water reported in Table 81, assuming the concentration of Na, Ca, and Cl to be halved.

166. If it were possible to remove the chlorides from the water reported in Table 81, would it be potable? Why?

CHAPTER XXVI

167. What was the turbidity of the water leaving the plain sedimentation basin at Youngstown, Ohio, from October, 1923,'to September, 1924, according to Table 82? (*b*) What was the turbidity of the same water leaving the coagulating basin?

168. Compute the periods of retention that are possible in the sedimentation basins shown in Fig. 222, making reasonable assumptions of necessary dimensions not shown or that cannot be scaled.

169. Derive an expression for the economical depth of an open circular settling basin in terms of height *h*, diameter *d*, capacity *Q*, and the cost of appurtenances *S*. It is assumed that the unit costs of all necessary items is known and is expressed literally.

170. What would be the economical depth, in feet, of a square reservoir with a capacity of 740,000 gal. if the cost of floor and roof, including columns, is $8 per sq. ft.; the cost of the wall, in dollars per foot of length, 0.8 times the square of the depth of water, in feet, in the reservoir; and the cost of pipes, valves, and other appurtenances, $1,000? There will be no excavation, backfill, or other items to be considered.

171. Three open rectangular sedimentation basins are to be constructed to allow periods of retention of 2, 4, 6, 8, and 10 hr., depending on the valves that are open or closed, when the flow through the entire plant is at the rate of 12 m.g.d. The basins are to be placed in a row, side by side, using a common dividing wall between basins. All basins are of the same length and depth. No excavation or backfill is to be considered. The floor will cost $5 per sq. ft.; exterior walls, in dollars per foot of length, as in Prob. 170; interior, 25 per cent more than exterior walls; all appurtenances, $3,000. Considering no other items, what would be the least amount of money for which these basins could be built?

172. Make a sketch of the basins in Prob. 171, and show, in diagrammatic form, the piping and valve arrangement that would permit the periods of retention shown. Number each valve, and show, in a table, the ones that would be open and closed for each period of retention.

CHAPTER XXVII

173. (*a*) How many square feet of floor space would be required to store 6 months supply of each chemical listed in Table 83 to permit the addition of 2 grains per gal. to a flow of 10 m.g.d.? (*b*) If the chemical is to be fed as a solution, what would be the volume of a solution storage tank for each of the chemicals listed in Table 83 to hold 8-hr. supply and to dose 2 grains per gal. to 10 m.g.d., assuming that the greatest recommended solution strength is prepared?

174. In the operation of a coagulation basin it is found that optimum coagulation is obtained at a pH of 6.8 at which point 0.95 grain of alum is required per gallon of water treated. Unfortunately, the natural pH of the water is 7.2, and when alum alone is added, 1.5 grains of alum is required per gallon of water treated. It is found by test that the addition of 0.5 grain of 66°Bé sulphuric acid will reduce the pH so that satisfactory coagulation can be obtained with 1.0 grain of alum per gallon of water treated. If the sulphuric acid costs 1 cent per lb. and the alum $1.50 per 100 lb., what will be the saving, if any, per million gallons of water treated when sulphuric acid is added? Answer in dollars.

175. How much natural alkalinity will be required in water to be coagulated if 2 grains of alum is used per gallon of water?

176. By how much is the sulphate hardness of water increased by coagulation with 1 grain per gal. of commercial "alum" containing 17 per cent aluminum sulphate?

177. In the coagulation of water with "lime and iron" 150 lb. of ferrous sulphate (crystallized) is added per million gallons of water. (*a*) How many pounds of commercial quicklime, containing 85 per cent CaO, must be added to complete the reaction, assuming no natural alkalinity in the water? (*b*) Is it more or less economical to add the iron first? Why? (*c*) If the water contains 12.0 p.p.m. of dissolved oxygen at the start of the reaction, how much will remain when coagulation is complete, assuming no reaeration or change of temperature during coagulation?

178. If 0.5 grain of pure aluminum sulphate is required per gallon of water treated, what would be the cost of commercial alum costing $2.00 per 100 lb. and containing 17 per cent aluminum sulphate, to treat 1,000,000 gal. of water? Answer in dollars.

179. The stirring paddles in a chemical mixing tank are so arranged that the shaft that drives them should revolve at 200 r.p.m. A two-phase 60-cycle 110-volt motor revolving at 1,800 r.p.m. is on hand. Only three-phase 60-cycle 110-volt current is available. The motor has a rated capacity of 5 hp. (*a*) If the cost of current is so low as to be negligible, what should you do in the situation? (*b*) If the current costs 4 cents per kw.-hr., what should you recommend? (*c*) If after installing the equipment you found the paddles revolving in the wrong direction, how would you reverse the motor?

180. The nominal capacity of a rapid sand filter plant is 1 m.g.d. Alum is used as a coagulant, the required dosage being 1 grain of alum per gallon of water treated. Three solution storage tanks are used, each tank holding sufficient solution for 8 hr. dosage. (*a*) What should be the capacity, in gallons, of each tank? (*b*) Of what materials should the tanks be made? Why?

181. If stirring paddles were installed in the solution-storage tanks in Prob. 180, how fast would the paddles revolve, in revolutions per minute; how large (diameter in feet) should they be; and what size (horsepower) electric motor would be required for driving the paddles in one tank? Assume each tank cylindrical in shape.

182. Determine the dimensions of an around-the-end baffled mixing basin for a flow of 10 m.g.d. and a retention period of 20 min. What would be the probable loss of head through this basin? What would be the difference between the electric power necessary to lift the water this height and the power necessary to drive flocculators that would give an equally satisfactory mix? (Suggestion: See *Eng. News-Record*, Vol. 99, p. 857; *Jour. A.W.W.A.*, Vol. 21, p. 235.)

183. Draw to scale, showing the dimensions of the pipes and fittings necessary, for a solution pot similar to Fig. 225, to hold sufficient alum to dose 2 m.g.d. of water at a rate of 2 grains per gal., assuming a recharge every 8 hr. Dimensions of pipes and fittings are to be taken from Table 56 in so far as it is possible. Other dimensions are to be taken from standards, from manufacturers' catalogues, or, where such information is not available, determined by yourself as designer.

184. Draw to scale a rectangular flocculation basin, with mechanically driven paddles, through which the water flows horizontally from one paddle to the next in series. The basin is to care for a flow of 1 m.g.d. with a flocculating period of 45 min. It is to be 8 ft. deep. Show over-all dimensions of the basins and details of inlet and outlet devices or arrangements, state nominal horizontal flowing-through velocity, and estimate the size (power) of the motor and the daily cost of electric power for operating the mechanism if the unit cost for power is 2 cents per kw.-hr. Use maximum power requirements given in Sec. 531. Assume all necessary data not given in the statement of the problem.

185. If the precipitator shown in Fig. 228 is 32 ft. in diameter at the top of the effluent collection trough, and it is treating a flow of 5 m.g.d., what would be the diameter in millimeters of the smallest particle of flocculent material removed within the "sludge filter zone" according to Stokes' law? The water temperature is 20°C., at which the viscosity of the water may be assumed as 1 centipoise. The density of the particle removed may be assumed as 1.05, where the density of water is 1.00 at 20°C. Stokes' law is $V = gd^2(D_1 - D_2)/18\mu$ in which V = settling velocity of particle in water in centimeters per second; g = acceleration of gravity in centimeters per second^2, d = diameter of settling particle in centimeters, D_1 = density of water in grams per centimeter^3, D_2 = density of settling particle in same units, and μ = coefficient of viscosity in poises. (*b*) If the settling particle has a density of 2.65, what would be the smallest size removed?

CHAPTER XXVIII

186. (*a*) What is the approximate nominal capacity of the filter shown in Fig. 230? (*b*) What would be the maximum velocity of flow in any of the branch underdrains?

187. If the depth of water on the sand filter shown in Fig. 231 is 50 in., how many inches of negative head can be produced at the surface of the sand?

188. A slow sand filter costs $125,000 per acre, exclusive of walls. The appurtenances to each filter unit cost $3,000. The cost of outside walls is $18 per foot, and the cost of walls common to two basins is $25 per foot. The cost of the administration building and other items common to the entire plant is $50,000. If the plant is to occupy 12 acres with rectangular units of equal area, laid out in the most economical fashion, what would be the least amount of money for which the plant could be built?

189. If the operator is called in emergency from the head house to the meter house at the filtered water basin, in Fig. 233, how long will the trip take him if he runs at 5 m.p.h.?

190. What should be the inside dimensions of a rectangular concrete rapid sand filter unit of the gravity type, capable of treating 1 m.g.d. at the standard rate of filtration? Show computations, and give reasons for the selection of each dimension.

191. What are the approximate width and length of the operating gallery shown in Fig. 235?

192. What is the maximum head loss possible between washings of the rapid sand filters in Fig. 236 when the clear well is full?

193. (*a*) At what rate must wash water be supplied to the filter unit shown in Fig. 238 if the rate of rise is 24 in. per min.? (*b*) What should be the dimensions of each lateral wash-water trough? (*c*) If the value of K in the Baylis formula in Sec. 548 is 60, what is the minimum time (length) of filter run on the unit shown in Fig. 238? Answer in hours.

194. A filter plant with a capacity of 12 m.g.d. is to include 12 rapid sand filter units of equal capacity, placed in two rows of six units each. Each filter unit is to be a monolith with all walls 12 in. thick. The filters are to be controlled by an automatic submerged rate controller, allowing a negative head as high as 4 ft. Explain all computations and steps necessary to determine the length, width, and depth of the inside of each filter box.

195. What are the effective size and the uniformity coefficient of the sand within the "specified limits" shown in Fig. 242?

196. Compute the following for a perforated-pipe underdrainage system branching from a manifold down the center of a 1 m.g.d. rapid sand filter unit, using wash water at 24-in rise per minute. The filter is 1.25 times as long as it is wide; there is a single manifold pipe down its center, with branches on 6-in. centers along the manifold. Determine (*a*) largest cross-sectional area of the manifold in square inches, (*b*) largest cross-sectional area of one branch in square inches, (*c*) diameter of the perforations in one lateral in inches, (*d*) number of such perforations, (*e*) total area of perforations in square inches.

197. Compute the cross-sectional area, in square inches, of the largest portion of the manifold and for each branch in a separate air-wash distribution system in a rapid sand filter unit of the size indicated in Prob. 196. There is a single manifold down the center of the filter with branches on 6-in. centers.

198. If the distance H in Fig. 217 is 10 ft., what percentage of the total head loss is represented by negative head?

199. If the diameter of the throat of the rate controller shown in Fig. 248 is 6 in., the diameter to the entrance of the balanced valve is 12 in., the velocity of flow through the throat is 16 f.p.s., and the diameter of the flexible diaphragm exposed to the differential pressure is 15 in., what is the magnitude of the force due to differential water pressure, and is it tending to open or to close the rate controller? (Neglect losses due to friction.)

200. Compute the velocity of flow in each of the main pipes in the pipe gallery in Fig. 236 assuming that there are eight filter units on each side of the gallery, all may be in normal operation at once, but only one filter may be washed at a time at a rate of 24-in. rise per minute.

201. A rapid sand filter plant, with 12 filter units, has a nominal capacity of 24 m.g.d. If water alone is used in washing and the wash water is supplied from a wash-water storage tank, what should be the capacity of the tank in gallons?

202. A rapid sand filter plant, with 18 filter units, has a nominal capacity of 24 m.g.d. If air and water are both used in washing, and the air is supplied from a wash-air storage tank, what should be the capacity of the tank in cubic feet?

203. What would be the cost of cleaning one filter shown in Fig. 236, with caustic soda and soda ash, according to the figures in Table 86, if the costs shown in Table 85 are in dollars per 100 lb. and NaOH costs $4 per 100 lb.?

204. If R in Streeter's formula, Sec. 588, is limited as stated in Sec. 589, what would be the value of E_a and in what units would it be expressed? Assume $T = 4$ hr.

CHAPTER XXIX

205. Examine *U.S. Geological Survey, Water Supply Paper* 496, and name one city that has a very soft water, one that has a moderately soft water, and one that has a water so hard as to be on the verge of rejection as a public water supply. In each case state the hardness of the water supply in parts per million and in grains per gallon.

206. How many pounds of lime and soda ash should be added per million gallons to soften the water in Table 81?

207. A Middle Western city with a population of 50,000 has a water supply with a total hardness of 400 p.p.m. What would be the annual savings per capita in the cost of soap if this water were reduced to a hardness of 100 p.p.m.? Base your conclusions on the following references: *Jour. A.W.W.A.*, June, 1932, p. 859. *Am. City*, Vol. 39, p. 131. *Jour. A.W.W.A.*, Vol. 12, p. 398; Vol. 14, p. 406. *Eng. News-Record*, Vol. 97, p. 780.

208. (*a*) How many pounds of lime, $Ca(OH)_2$, must be added to 1 million gal. of water to reduce the carbonate hardness from 200 p.p.m. to 100 p.p.m. according to reactions (1) and (2) in Sec. 596? (*b*) If the water were saturated with CO_2 at a temperature of 60°F. when the lime is added, what would be the percentage saturation after the reaction is complete? (*c*) If recarbonation is applied at the maximum rate stated in Sec. 599, so that saturation with CO_2 is restored, how many pounds of CO_2, per million gallons, escape to the atmosphere?

209. Five million gallons of water, containing 250 p.p.m. total hardness, is to be softened daily through greensand zeolite, through which the water passes downward by gravity. (*a*) Plot a curve with cubic feet of zeolite as ordinates and period between regeneration in hours as abscissas, up to a maximum period of 1 week. Make no allowance for any excess zeolite for any purpose. (*b*) How many pounds of salt would probably be used weekly in the operation of this plant?

CHAPTER XXX

210. (*a*) How many pounds of chlorine gas will be required to apply a dose of 0.2 p.p.m. to a million gallons of water? (*b*) How many pounds of calcium hypochlorite (commercial) would be required for the same purpose? (*c*) How many pounds of H-T-H would be required for the same purpose?

211. (*a*) How many parts per million of chlorine must be added to the water in Fig. 256 to obtain a residual of 1.5 p.p.m.? (*b*) If 2.5 p.p.m. of chlorine is added, what will be the residual? (*c*) If chlorine costs 1.5 cents per lb., ammonia costs 3 cents per lb., ammonia is to be added at a ratio to chlorine of 1:3 and it is desired to produce a residual of 1 p.p.m. of chlorine as indicated by the orthotolidine test, what would be the saving in cost of chemicals, per million gallons of water treated, if break-point chlorination were used instead of applying the chlorine-ammonia treatment?

212. How much electric power, in kilowatt-hours would be required for the sterilization of 1,000,000 gal. of water by ultraviolet light? The following references may be of assistance: *Jour. A.W.W.A.*, Vol. 1, p. 564; Vol. 4, p. 172; Vol. 7, p. 325; Vol. 18, p. 775; Vol. 22, p. 959; Vol. 22, p. 1290. *Eng. News-Record*, Vol. 79, p. 1021. *Jour. New Engl. Water Works, Assoc.*, Vol. 29, p. 208. *Am. Jour. Pub. Health*, Vol. 12, p. 320. *Eng. Contr.*, Vol. 57, p. 46.

213. How much electric power, in kilowatt-hours, would be required for the sterilization of 1 million gal. of water by ozonization, according to the figures at Ashton shown in Table 89?

CHAPTER XXXI

214. Water issuing from a well contains no dissolved gases and 4 p.p.m. of ferrous hydroxide. (*a*) How many pounds of oxygen must be dissolved to oxidize 90 per cent of the ferrous hydroxide to ferric hydroxide? (*b*) How much time will be required for the absorbtion of the oxygen by the water? (Suggestion: See *Jour A.W.W.A.*, Vol. 24, p. 65. Explain your conclusion.)

215. A body of water at a temperature of 50°C. is 3 ft. deep and 10 ft. square, with a smooth, unbroken surface exposed to the air. How long will be required for the percentage of saturation of oxygen to rise from 20 to 90? (Suggestion: See *Jour. A.W.W.A.*, Vol. 24, p. 61.)

216. Plot curves with efficiencies as ordinates and concentration of CO_2 as abscissas, for various types of aerators. Efficiencies are to be measured in percentage removal of CO_2. A curve is to be drawn for each type of aerator studied. (Suggestion: Consult *Eng News-Record*, Vol. 90, p. 874. *Jour. A.W.W.A.*, Vol 11, p. 118. *Pub. Works*, Vol. 55, p. 251. *Eng. Contr*, Vol. 40, p 302. *Water Works & Sewerage*, August, 1932, p. 275)

217. The tabulated figures show the changes in alkalinity and in pH resulting from the addition of calcium carbonate to 1 l. of water. What are the alkalinity and pH of the water at McLaughlin's saturation point? How many pounds of lime, $Ca(OH)_2$, must be added to 1 million gal of this water to attain carbonate balance?

Alkalinity	115	130	142	152	158	160	158	154	148
pH.	4.3	5.3	6.4	6.8	7.2	7.7	8.0	8.4	9.0

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